VOL. XXX January, 1952 No. 1

THE STRUCTURAL ENGINEER

THE JOURNAL OF THE INSTITUTION OF STRUCTURAL ENGINEERS

Contents

3s. 6d.

VOL. XXX No. 1 JANUARY, 1952

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Founded 1908. *Incorporated by Royal Charter* 1934.

11 UPPER BELGRAVE STREET, LONDON, S.W.1. Sloane 7128/9.

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VOL. XXX No. 2 FEBRUARY, 1952

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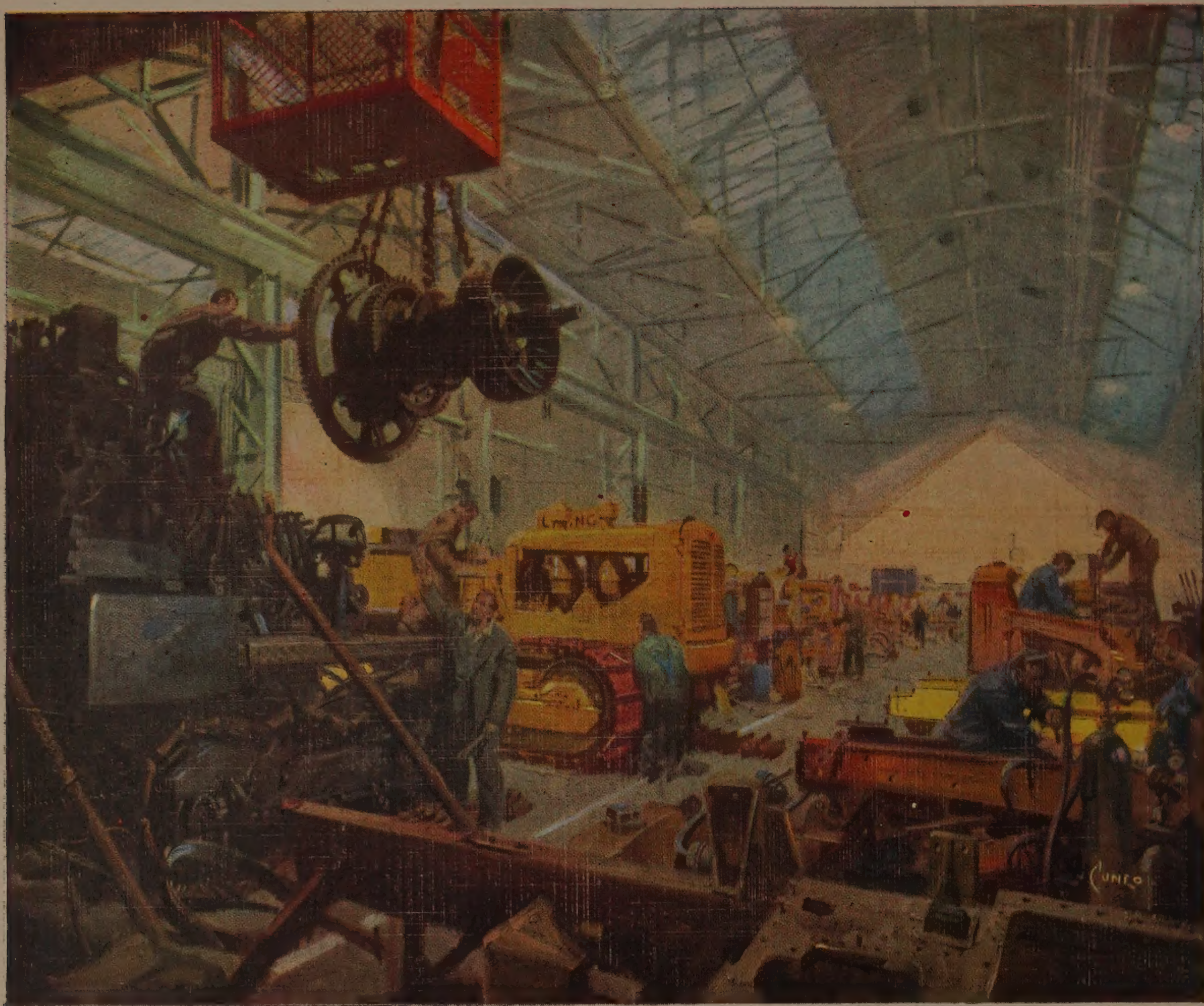

VOL. XXX No. 3 MARCH, 1952

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VOL. XXX No. 5 MAY, 1952

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VOL. XXX No. 9 SEPTEMBER, 1952

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VOL. XXX No. 11 NOVEMBER, 1952

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VOL. XXX No. 12 DECEMBER, 1952

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A New Approach to the Elastic Analysis of Two Dimensional Rigid Frames*

By Arthur Bolton, M.Sc.Tech.(Graduate)

Summary

Slope Deflection, Moment Distribution, and Relaxation are the most widely used methods of solution of rigid frames, but each requires a great deal of labour for larger structures. Possibly the main cause of this difficulty is the need to consider two types of deformation of the structure, rotation of joints and sways. At present these are dealt with by considering them as separate unknowns.

This paper shows that it is possible to consider the effects of either a rotation or a sway as a pattern of bending moments and forces. Thus there is no need to consider two types of unknown, but the solution can be carried out entirely in terms of one. This new concept of the unknowns reduces enormously the time required for any solution.

Each of the methods mentioned solves the same basic equations, but in different ways, having advantages and disadvantages compared with the others. By suitable combination of parts of different methods a further saving of labour is obtained. Moreover, solution of one problem on a structure in this way will yield the greater part of the solution of any other problem on the same structure. Hence it becomes economically possible to consider many load cases on a large structure.

It is expected that the method described will greatly reduce the time and labour required for the design of structures and that this saving will increase in proportion to the complexity of the structure.

Introduction

The development of the analysis of rigid frames in recent years has been as follows :—

1. Strain Energy showed that the problems could be reduced to obtaining the solution of a set of simultaneous linear equations ; gave methods of finding the necessary equations ; and showed that a sufficient number of equations could always be obtained.[1]

2. Within the limitation that deflections due to shear forces and axial forces were ignored, Slope Deflection systematised and simplified the equations, and enabled the first practical solutions of certain problems to be obtained. By taking one member at a time and considering the end slopes and deflections a more easily visualised system of unknowns was obtained.[2]

3. Once the problem had been reduced to that of solving a set of simultaneous linear equations a start was made on improving mathematical methods, leading to solution by Remainder Distribution and Matrices.[3]

4. At the same time Type Solutions were considered. Some general and many special case solutions were obtained. The use of symmetrical and anti-symmetrical loading, etc., extended the range of problems which could be solved.[4]

5. An entirely different outlook was given by Moment Distribution. The problem was attacked in a series of physical steps, each of which could be calculated, the exact solution was approached as closely as required by successive corrections until the range of uncertainty was within the limits of uncertainty of the data.[5]

6. Relaxation again considered a series of physical steps but considered the solution in terms of displacements of the structure. It is much more general and powerful than Moment Distribution and allows much more freedom of procedure. Apart from strain energy it is the only method which can take account of shear and axial forces.[6]

Of these methods Moment Distribution has attracted the practical engineer and has hardened into a routine which may be used automatically by draughtsmen. Whilst this is very convenient from some points of view it is unfortunate from others since a great deal of the tedious arithmetic implicit in routine solutions can often be avoided. In particular, the very powerful methods of Relaxation have not been popularised, although they can be used with advantage in ordinary structural problems.

Consideration of the Various Methods

Slope Deflection and Type Solutions

The standard method of Slope Deflection is to express the bending moments at the ends of members in terms of rotations and sways. The ruling formula is :—

$$M_{AB} = \frac{2EI}{L}\left[2\theta_A + \theta_B - \frac{3\delta}{L}\right] + MF_{AB}$$

where MF_{AB} is the fixed end moment due to any loads in span AB. Positive displacements are indicated in Fig. 1, and clockwise moments at joints are accounted positive.

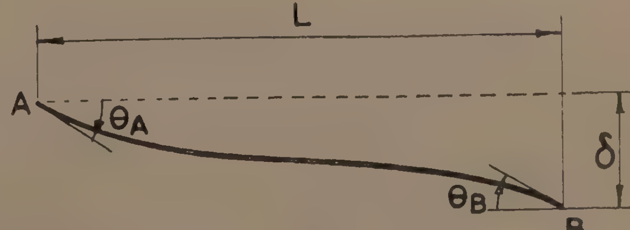


Fig. 1

The equations used are the equations of equilibrium of the joints, and the shear equations. The unknowns solved for are the rotations and deflections of the joints. Finally the moments are obtained by substituting these slopes and deflections in the above formula.

*Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, January 24th, 1952, at 6 p.m.

The disadvantages, from the calculator's point of view, are the conversion from moments to slopes and back again to moments, and the difficulty of solving the simultaneous equations with sufficient accuracy.

Type Solutions obtained by use of the same formula can also be used to solve these problems. The simplest Type Solutions are those for rotation of each joint, with all adjacent joints fixed, and for the sway of each storey. The solution of the problem is obtained by combining the Type Solutions in such proportions that the conditions for equilibrium are satisfied.

These two methods may be illustrated by a simple example.

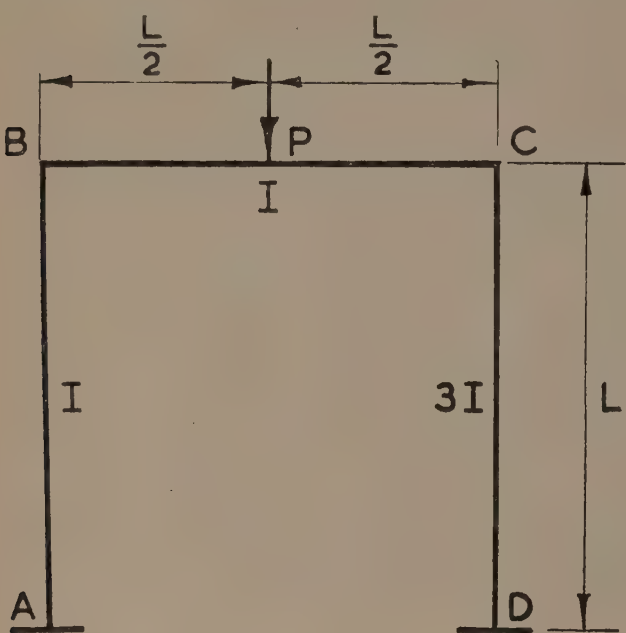


Fig. 2

Original Formulae

$$M_{AB} = \frac{2EI}{L}\left(\theta_B - \frac{3\delta}{L}\right)$$

$$M_{BA} = \frac{2EI}{L}\left(2\theta_B - \frac{3\delta}{L}\right)$$

$$M_{BC} = \frac{2EI}{L}\left(2\theta_B + \theta_C\right) - \frac{P.L}{8}$$

$$M_{CB} = \frac{2EI}{L}\left(\theta_B + 2\theta_C\right) + \frac{P.L}{8}$$

$$M_{CD} = \frac{6EI}{L}\left(2\theta_C - \frac{3\delta}{L}\right)$$

$$M_{DC} = \frac{6EI}{L}\left(\theta_C - \frac{3\delta}{L}\right)$$

Joint and Shear Equations

Equilibrium of Joint B. $M_{AB} + M_{BC} = 0$

Therefore $4\theta_B + \theta_C - \frac{3\delta}{L} - \frac{P.L^2}{16EI} = 0$. (1)

Equilibrium of Joint C. $M_{CB} + M_{CB} = 0$

Therefore $\theta_B + 8\theta_C - \frac{9\delta}{L} + \frac{P.L^2}{16EI} = 0$. (2)

Shear Equation. $M_{AB} + M_{BA} + M_{CD} + M_{DC} = 0$

Therefore $3\theta_B + 9\theta_C - \frac{24\delta}{L} = 0$. (3)

$3\times(1) - (2) \quad 11\theta_B - 5\theta_C - \frac{P.L^2}{4EI} = 0$. (4)

$8\times(1) - (3) \quad 29\theta_B - \theta_C - \frac{P.L^2}{2EI} = 0$. (5)

$5\times(5) - (4) \quad 134\theta_B - \frac{9P.L^2}{4EI} = 0$

Therefore $\theta_B = \frac{9}{536} \cdot \frac{P.L^2}{EI}$

$\theta_C = -\frac{7}{536} \cdot \frac{P.L^2}{EI}$

$\frac{3\delta}{L} = -\frac{9}{1072} \cdot \frac{P.L^2}{EI}$

These are the values which are substituted in the original formulæ to obtain the final solution.

Final Solutions

$$M_{AB} = 2\left[\frac{9}{536} + \frac{9}{1072}\right]P.L = +\frac{27}{536}P.L$$

$$M_{BA} = 2\left[\frac{18}{536} + \frac{9}{1072}\right]P.L = +\frac{45}{536}P.L$$

$$M_{BC} = 2\left[\frac{18}{536} - \frac{7}{536}\right]P.L - \frac{P.L}{8} = -\frac{45}{536}P.L$$

$$M_{CB} = 2\left[\frac{9}{536} - \frac{14}{536}\right]P.L + \frac{P.L}{8} = +\frac{57}{536}P.L$$

$$M_{CD} = 6\left[-\frac{14}{536} + \frac{9}{1072}\right]P.L = -\frac{57}{536}P.L$$

$$M_{DC} = 6\left[-\frac{7}{536} + \frac{9}{1072}\right]P.L = -\frac{15}{536}P.L$$

Slope Deflection with Type Solutions

To simplify the presentation, the values of the bending moments are shown on drawings of the structure.

The Type solutions taken for this problem are :

(a) Rotation of B with C fixed

(b) Rotation of C with B fixed

(c) Sway of the whole frame with BC maintained horizontal.

Using the Slope Deflection formulæ these are seen to give the pattern of bending moments in Fig. 3.

It is these three patterns which must be combined to obtain equilibrium with the fixed end moments produced by the load. Since this is so, it is necessary to consider only the proportions of bending moment in each Type Solution. The correct amount of each Type Solution which satisfies the problem is automatically obtained in the later calculations.

The preliminary Type Solutions may thus be written in a proportional form as in Fig. 4.

The problem then resolves itself into combining (a), (b), and (c) in such proportions that the fixed end

$$8\times(1) - (3)\quad 29x - y - PL = 0 \quad . \quad (5)$$

$$5\times(5) - (4)\quad 134x - \frac{9PL}{2} = 0 \quad .$$

$$\text{Therefore}\quad x = \frac{9PL}{268}$$

$$y = -\frac{7PL}{268} \qquad z = \frac{9PL}{536}$$

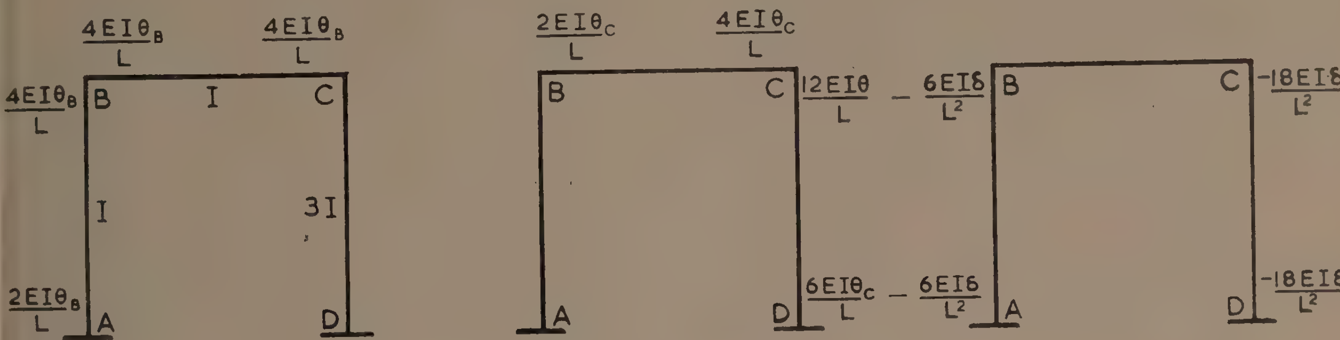


Fig. 3

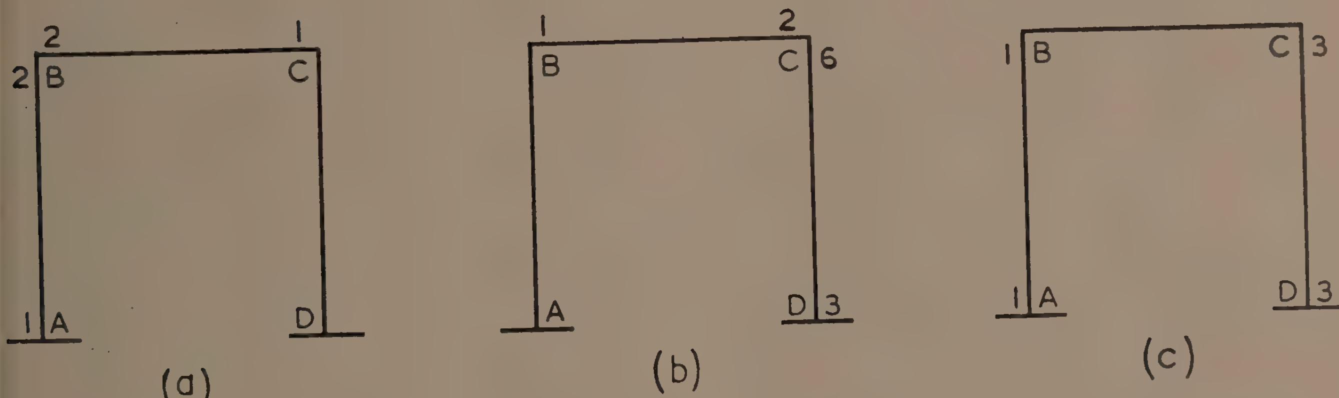


Fig. 4

moments due to the load are cancelled. If these proportions are assumed to be x times (a), y times (b) and z times (c), the equations of equilibrium become:—

$$\text{Joint B.}\quad x(2+2)+y(1)+z(1) - \frac{PL}{8} = 0 \quad . \quad (1)$$

$$\text{Joint C.}\quad x(1)+y(2+6)+z(3)+\frac{PL}{8} = 0 \quad . \quad (2)$$

$$\text{Shear Equation.}\quad x(2+1)+y(6+3) + z(1+1+3+3) = 0 \quad . \quad (3)$$

$$\text{i.e.,}\quad 4x+y+z - \frac{PL}{8} = 0 \quad . \quad (1)$$

$$x+8y+3z + \frac{PL}{8} = 0 \quad . \quad (2)$$

$$3x+9y+8z = 0 \quad . \quad (3)$$

$$3\times(1) - (2)\quad 11x - 5y - \frac{PL}{2} = 0 \quad . \quad (4)$$

Therefore

$$M_{AB} = \frac{9PL}{268}(1) + \frac{9PL}{536}(1) = +\frac{27P.L}{536}$$

$$M_{BA} = \frac{9PL}{268}(2) + \frac{9PL}{536}(1) = +\frac{45P.L}{536}$$

$$M_{BC} - \frac{9PL}{268}(2) - \frac{7PL}{268}(1) - \frac{PL}{8} = -\frac{45PL}{536}$$

and so on for the other members.

It will be seen that the form of the simultaneous equations is similar in each case. The static checks for equilibrium that $M_{AB} + M_{BA} = 0$, and that the sum of the Column End Moments equals the shear times the storey height, are still present.

There are two advantages in the second method of presentation. It affords a visual representation of the moments due to each operation of rotation and sway, which enables the three simultaneous equations to be written down with less possibility of error. It also leads to simplification, even more pronounced in complex structures, since apart from demonstration, Fig. 3 is never needed and the problem is started by writing down the patterns of Fig. 4.

Moment Distribution

The standard method is the solution step by step of partial problems. In the first place, if all the joints were fixed against translation and rotation by external constraints the solution could be found. But in the above problem, for example, B is not fixed. Therefore B is allowed to rotate balancing out the fixed end moment at B. This produces a redistribution of the bending moments at B, and by carry-over gives half the appropriate balancing moment at the other end of members AB and BC. All these can be calculated.

There will remain an out-of-balance moment at C. If B is now fixed and C allowed to rotate, a similar action takes place.

The structure now requires a horizontal force at B or C to maintain equilibrium. If desired, the structure may be allowed to sway such an amount without rotation of B or C that this force becomes zero. This occurs when :

$$M_{AB} + M_{BA} + M_{CD} + M_{DC} = 0$$

and so this partial problem can also be solved.

At the end of this first cycle, therefore, by allowing B and C to rotate and the whole structure to sway sideways, the external constraints will have been greatly reduced. There will still remain out-of-balance moments at B and C, but by continuing the process as accurate a solution as desired may be obtained.

By considering these steps individually it is seen in fact that Moment Distribution uses the simple type solutions of Fig. 3, but solves by physical successive correction. The limitation of Moment Distribution is that the procedure is automatic and convergence may be slow in certain problems.

Moreover, Moment Distribution does not carry the idea of Type Solutions to its conclusion but solves each particular load case as a separate problem. Example (3) (see later) demonstrates how much more quickly the solution to a number of loading cases on one structure can be obtained.

Relaxation

The procedure applied to frames is to give unit displacements, called operations, at each joint in turn and calculate the values of moments and forces produced. For two dimensional frames there are two displacements and a rotation at each joint producing moments shears, and axial forces in the several members. The basic method of solution is then to relax the external constraints for the fixed end solution using combinations of the various operations.

The advantages of Relaxation are :

1. It is very powerful and can solve many problems otherwise insoluble.

2. It is flexible and can be used in different ways to attack different problems.

3. It is a method of successive approximation as opposed to successive correction. The operator is allowed to use his knowledge and previous experience of similar problems to speed the solution. He may even choose to estimate the solution, and in effect start with the problem almost finished without any loss of accuracy.

4. Extra redundancies or members add little to the labour of solution.

5. Any further problems on the same structure can be solved with little extra labour.

The disadvantages of Relaxation are :

1. There is a great deal of tedious arithmetic in calculating the effects of the unit displacements.

2. There is again the labour of starting with moments, working in terms of displacements, and converting back to moments.

3. An error which would be obvious in terms of moments may pass unrecognised in terms of rotations and deflections.

Synthesis of a New Method of Calculation

It can be seen that if only the disadvantages of Relaxation could be avoided it would be a most suitable method for attacking structural problems. This can be accomplished with some loss of generality.

For instance, in many problems extensions of the members and deflections due to shear may be neglected (as is implicit in Moment Distribution and Slope Deflection) and this saves a great deal of labour in calculating the effects of the remaining unit displacements. Moreover, the second and third disadvantages of Relaxation can now be avoided by working throughout in terms of moments and forces, using patterns as obtained for the Type Solution method described. Fig. 4 only shows the moments, but the horizontal force associated with each pattern can be obtained by dividing the sum of the column end moments by the storey height.

Constraints at a joint may be of two types, against rotation and against displacement. In problems where sway need not be considered it is only necessary to keep track of the constraints against rotation. Where this is so only the moment patterns need to be considered. Many problems involving sway can also be reduced to the consideration of moment patterns only, by the use of " no shear " patterns as will be explained later.

The amount of either the external constraints, or the internal bending moments in the members, is retained at each step in the calculation. For sway problems where the patterns require the consideration of forces it will probably be found advantageous to abstract the force from the moment patterns in tabular fashion.

In using the new method it has been found desirable to combine the required characteristics from amongst the methods outlined. Thus, a tabulation similar to that of Moment Distribution, the Slope Deflection formula for obtaining the patterns of bending moment for the various displacements, building up and combining these patterns to obtain more complex operations as in Relaxation, and the methods of Type Solutions are used to speed the solution.

Summary of the Method

The method may be summarised as follows :—

Step 1. With all joints fixed against rotation and deflection the values of the various constraints, e.g., fixed end moments and reactions, are calculated.

Step 2. The effects of all possible rotations and sways are written down one by one in tabular form, or on sketches of the structure as indicated before. For

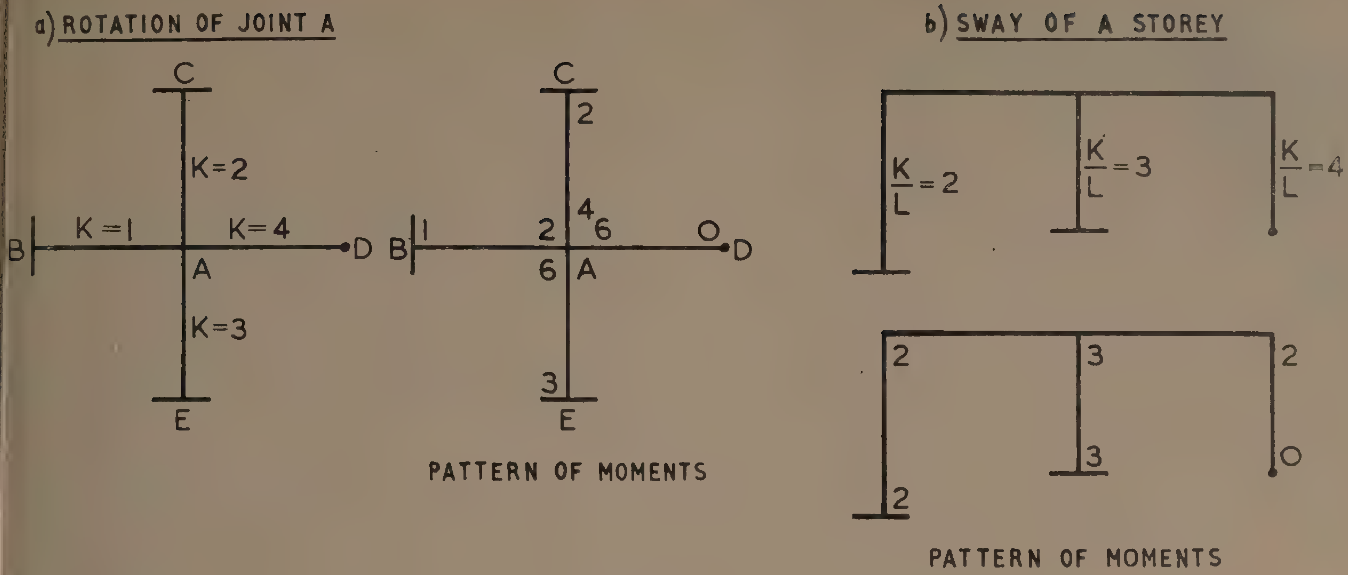


Fig. 5

instance, in the portal of Fig. 2 there will be three such operations—rotation of B, rotation of C, and sway of B and C. These operations are obtained in proportion as before, with the addition of the horizontal external force required, which is not shown in Fig. 4.

Two expressions only, obtained from the Slope Deflection formula, are needed to calculate these operations for frames with no inclined members.

(a) For rotations. With fixed ends $M_{AB} \propto K_{AB}$ and the carry-over factor is a half.

(b) For sways $M_{AB} = M_{BA} \propto \dfrac{K_{AB}}{L_{AB}}$, where K the relative stiffness is proportional to $\dfrac{I}{L}$ and L is the length of the member considered.

Fig. 5 gives examples of such operations.

Step 3. These separate operations are combined to comply with the known facts of the problem. For instance, in symmetrical problems the rotations of corresponding joints will be the same.

Step 4. The combined patterns so obtained are used to relax the fixed end constraints. This may be done in three ways.

(a) In simple problems by the solution of simultaneous equations or by inspection.

(b) By a process of working out distribution and carry-over coefficients for the combined operations and using them in a similar way to Moment Distribution.

(c) By the normal relaxation process following R. V. Southwell.

The choice of method will usually be obvious when this step is reached.

The various processes have been illustrated by the examples which follow. It has not been possible, however, with so few examples, to demonstrate fully the flexibility of this kind of attack.

The value of the method is thought to be in the extension of the field of problems which can be easily solved. Its utility may be judged by comparison with other methods of solution.

Example 2

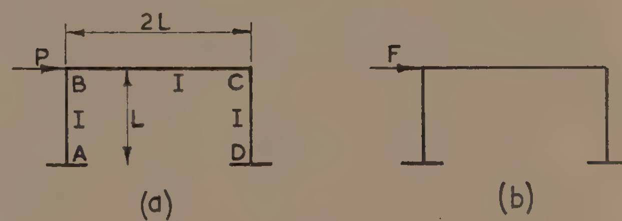


Fig. 6

1. All fixed end moments are zero.
2. Table of moments caused by the various operations

	Operation	A	B	B	C	C	D	$F \times L$
(a)	Rotation of B...	2	4	2	1			—6
(b)	Rotation of C ...			1	2	4	2	—6
(c)	Sway	1	1			1	1	—4

F is the external horizontal force acting along BC caused by the operation considered, and is taken to be positive in the direction shown in Fig. 4 (b). It is found by dividing M_{AB}, M_{BA}, M_{CD} and M_{DC} by the respective stanchion lengths and adding.

3. In this symmetrical problem $\theta_B = \theta_C$, therefore operations (a) and (b) must be added in the same proportion.

(d)	(a) + (b) ...	2	4	3	3	4	2	—12

4. But for equilibrium $M_{BA} + M_{BC} = 0$ and since the moment constraint at B due to operation (d) is $+ 7$ units the sway operation must cause -7 units at B.

(e)	(d) —7 (c)	—5	—3	+3	+3	—3	—5	+16

This gives the proportion of moments for this problem, but the size has yet to be fixed. However, the shear equation may be used, PL corresponding to $+16$.

$$\text{Therefore } M_{AB} = M_{DC} = -\frac{5}{16}PL$$

$$-M_{BA} = M_{BC} = M_{CB} = -M_{CD} = \frac{3}{16}PL$$

Example 3

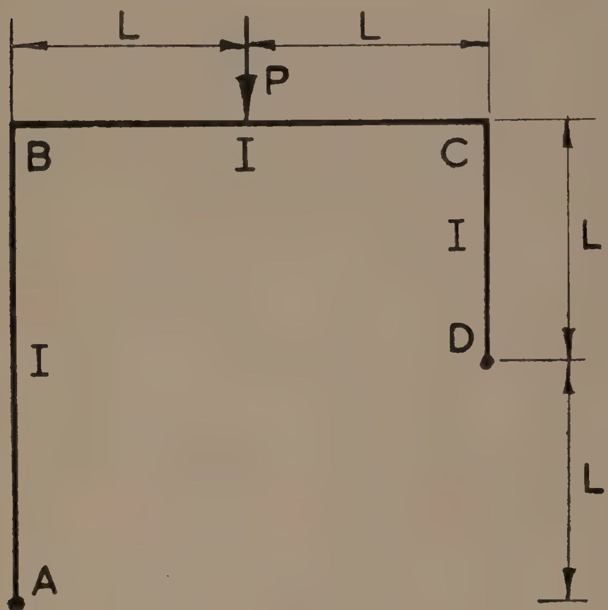


Fig. 7

1. Fixed end moments—$M_{BC} = M_{CB} = \frac{PL}{4}$

2.

	Operation	Pattern of Bending Moment produced						$F \times L$
		A	B		C		D	
(a)	Rotation of B...	0	3	4	2	0	0	—1.5
(b)	Rotation of C...	0	0	1	2	3	0	—3
(c)	Sway	0	1	0	0	4	0	—4.5

3. It is known that there is no horizontal constraint provided to maintain equilibrium. Therefore $F = 0$.

Operations (a) and (b), and (a) and (c) are combined to make this so.

(d)	(a) × 2 —(b) ...	0	6	7	2	—3	0	0
(e)	(a) × 3 —(c) ...	0	8	12	6	—4	0	0

4. Operations (d) and (e) can be used to relax the fixed end moments for any position of the load. More general expressions can be obtained by combining (d) and (e) to obtain two operations, in one case with $M_B = 0$ and in the other with $M_C = 0$.

To obtain them the total moments M_B and M_C in (d) and (e) are considered.

In (d) $M_B = 13$ $\quad M_C = -1$
In (e) $M_B = 20$ $\quad M_C = +2$

(f)	(d) × 10 —(e) × 6½	0	+8	—8	—19	—4	0
(g)	(d) + (e) × ½	0	+10	+13	+5	—5	0

Operation (f) will relax $+23$ units of BM at C without affecting B.

Operation (g) will relax -23 units of BM at B without affecting C.

When obtained in the lowest multiple form it will always be found that pairs of operations like (f) and (g) relax the same amount of bending moment numerically at B and C.

It is of interest to note that so far only the properties of the structure have been used in the calculation ; the moments caused by the load have not yet been considered.

In the problem given $F.E.M.$ at B $= -F.E.M.$ at C.

Therefore the operation which will relax the $F.E.M.$s is $f + g$.

(h)	(f) + (g) add F.E.M.	0	+18	+ 5 —23	—14 +23	—9	0
	Final B.M.s	0	+18	—18	+9	—9	0

But the original $F.E.M.$ was $\frac{PL}{4}$ not 23.

Therefore

$$M_{BA} = -M_{AC} = 18 \times \frac{PL}{4 \cdot 23} = \frac{9}{46}PL$$

$$M_{CB} = -M_{CD} = 9 \times \frac{PL}{4 \cdot 23} = \frac{9}{92}PL$$

If the solution is required for some other loading case, say with the load at a quarter point as in Fig. 8, operations (f) and (g) are still used.

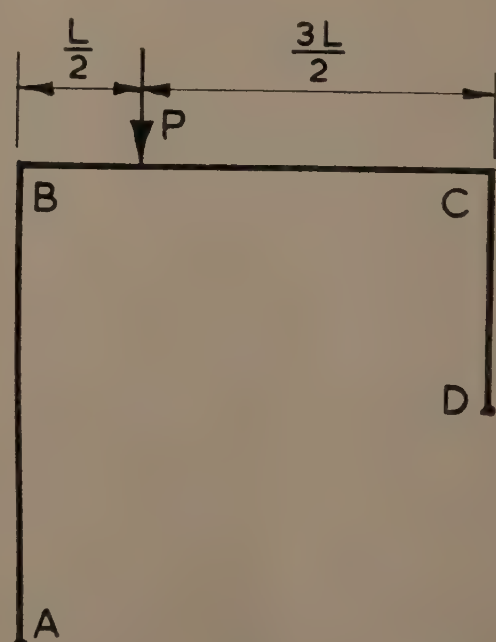


Fig. 8

1. $F.E.M.$s $M_{BC} = -\frac{9}{32}PL \quad M_{CB} = +\frac{3}{32}PL$

4. The appropriate operation will now be (f) + 3 × (g) since three times as much moment is to be relaxed at B.

(f) + 3 × (g) Add F.E.M.	o +38	+31—4 —69+23	—19 o	
Final B.M.s	o +38	—38+19	—19 o	

But 69 corresponds to $\frac{9}{32} PL$

Therefore

$$M_{BA} = -M_{BC} = 38 \times \frac{9PL}{32.69} = \frac{57}{368} PL$$

$$M_{CB} = -M_{CD} = 19 \times \frac{9PL}{32.69} = \frac{57}{736} PL$$

Likewise any other loading case could be quickly solved.

Alternatively by using general expressions for the *F.E.M.*s the influence lines for M_{BC} and M_{CB} could be obtained.

Example 4

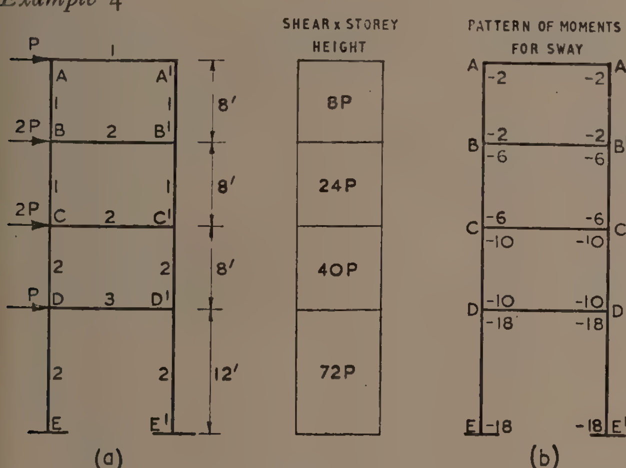


Fig. 9

The figures against the members of the frame in Fig. 9 (a) denote the relative stiffnesses. These are over-simplified for a practical case where fractional values would be required. This involves very little extra labour, but the derivation of the following patterns is more clearly seen when whole numbers are used.

1. All fixed end moments are zero.

2. In this case it is expedient to write the patterns of bending moment on drawings of the structure.

As the structure is symmetrical, A and A′ for instance will rotate the same amount. Therefore rotation of both joints can be dealt with on one pattern.

Patterns for Equal Rotation of A and A′, B and B′, etc.

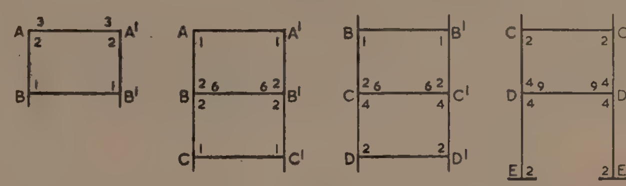


Fig. 10

The figure "3" for the moment AA′ in the first pattern is made up of 2 units from the rotation of A, and 1 carried over from the rotation of A′, and similarly with BB′, CC′ and DD′.

Pattern for Sway in any Storey

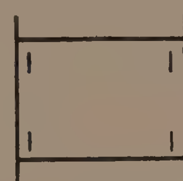


Fig. 11

3. One way of tackling this problem is to allow the structure to sway without any joint rotations giving the moment pattern of Fig. 9b. This gives a pattern

TABLE 1

	E	D			C			B			A	
		L	B	U	L	B	U	L	B	U	L	B
Dist. Coef.		.09	.82	.09	.13	.80	.07	.07	.86	.07	.14	.86
Sway B.M.s	—18	—18	o	—10	—10	o	—6	—6	o	—2	—2	o
Balance	o	+2.52	+22.96	+2.52	+2.08	+12.80	+1.12	+.56	+6.88	+.56	+.28	+1.72
Carry Over	—2.52	o		—2.08	—2.52		—.56	—1.12		—.28	—.56	
Balance	o	+.18	+1.72	+.18	+.40	+2.48	+.20	+.10	+1.20	+.10	+.08	+.48
Carry Over	—.18	o		—.40	—.18		—.10	—.20		—.08	—.10	
Balance	o	+.04	+.32	+.04	+.04	+.22	+.02	+.02	+.24	+.02	+.01	+.09
Carry Over	—.04											
Total B.M.	—20.74	—15.26	+25.0	—9.74	—10.18	+15.50	—5.32	—6.64	+8.32	—1.68	—2.29	+2.29

which complies with the shear equation but, which has an out-of-balance moment at each joint. These out-of-balance moments can be relaxed by rotations but the total of the column end moments for each storey must not be altered. Hence the required " no shear " patterns are those in which the sum of the column end moments is zero. In the pattern for rotation of A and A′ for instance in Fig. 10, the sum of the column end moments is 6. In the pattern for sway Fig. 11, the sum of the column end moments is 4. Therefore $1\frac{1}{2}$ times the sway pattern must be subtracted from the first rotation pattern, giving the " no shear " pattern of Fig. 12.

Combined " No Shear " Patterns

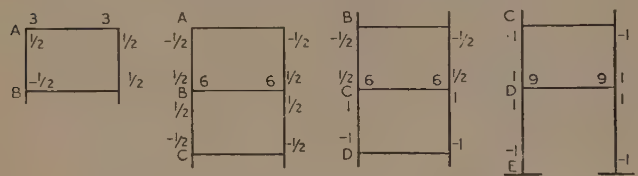


Fig. 12

From these " no shear " patterns the distribution coefficients are calculated. For example, at joints A and A′ the out-of-balance moment will be distributed so that $\frac{3}{3\frac{1}{2}}$ or .86 is carried by the beam.

The carry-over factor for the stanchions is seen to be —1 in each case.

4b. When the distribution coefficients have been obtained the sway moments of Fig. 9b are entered as the first line of a normal Hardy Cross type distribution in Table 1. If P is measured in tons the final line of Table 1 should be multiplied by P to give the solution in tons feet.

Example 5 (Fig. 13)

The structure of Fig. 13 (a) is symmetrical about its vertical centre line, so only half of it needs to be considered. The figures against the members in Fig. 13 (a) denote the relative stiffnesses. The wind load case has been chosen because in general this is the most difficult case to solve. In a practical problem it would be as easy and probably more expeditious to work in decimal distribution coefficients from the start. So that the building-up process may be more clearly demonstrated, however, care has been taken to keep the patterns in whole numbers.

1. All $F.E.M.$s are zero.
2. Patterns for rotations and sways.
3. " No Shear " patterns are obtained as for Example 4 by combining the patterns for rotation and sway of Fig. 14. The " no shear " patterns are set out in Fig. 15 firstly as whole numbers, and secondly in decimals so that the bending moment at the point under consideration totals unity.

If the Hardy Cross method is followed the bending moments in each member intersecting at a joint are written down at each stage of the calculation. Alternatively following Southwell, the number of applications of each pattern required to liquidate the constraints at the joints may be noted and finally converted into the internal bending moments.

For simpler problems, keeping account of the internal moments is better, but the second approach admits of

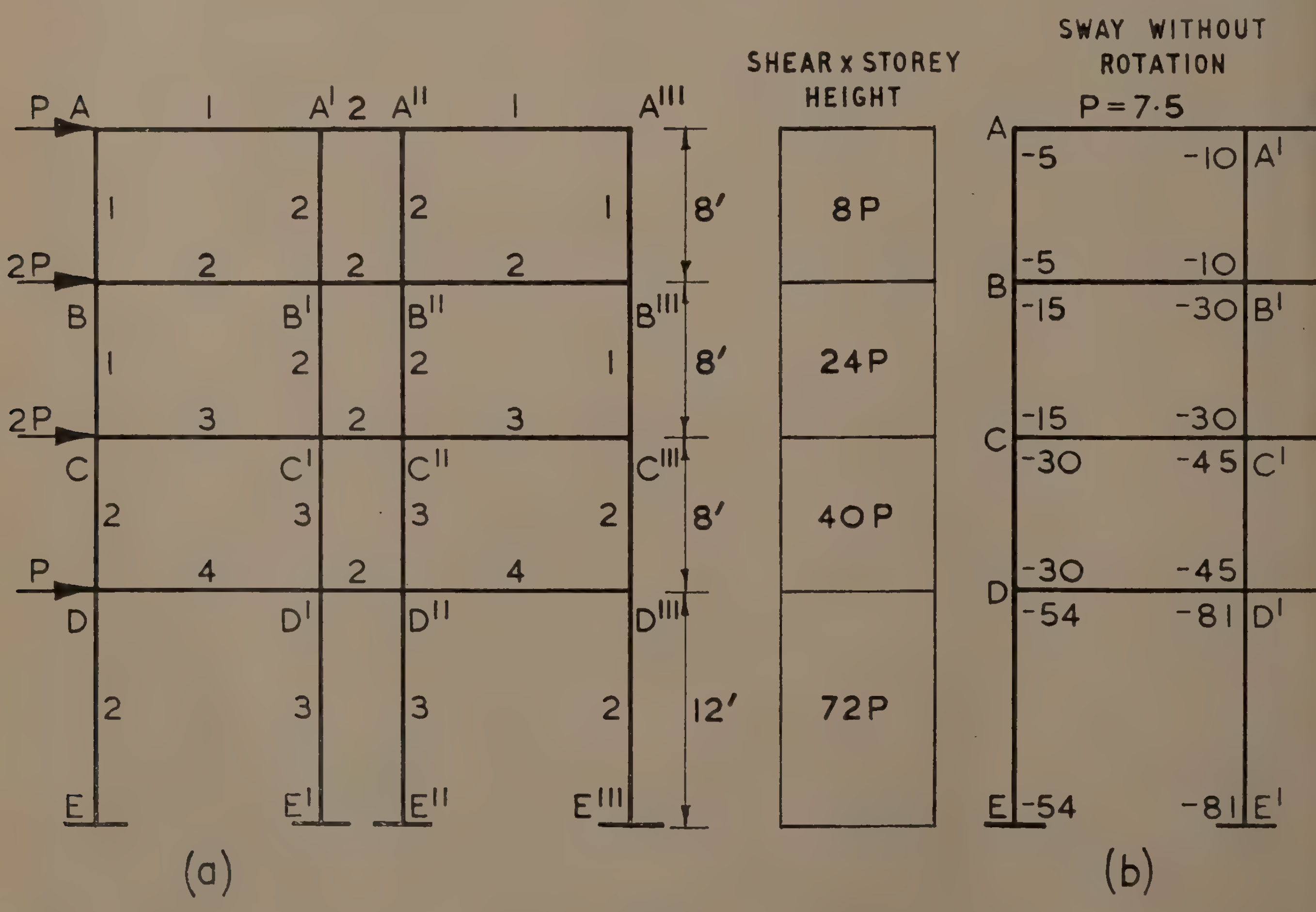


Fig. 13

dealing with complicated frameworks in a manner already familiar as the grid method of relaxation applied to partial differential equations.

(4b) Table 2 sets out the solution which is very similar to that of Example 4. In this case, however, there is a carry-over across the beams as well as the stanchions.

A′ was the first joint balanced after the sway moments of Fig. 13 (b) had been entered in the table. There is a balancing moment of +10 units required at A′. Using

In the second cycle the larger out-of-balance moments were attacked first, in the order C′, C, D′, D, B′, B, A′, A. At this stage the largest residual was about 1 per cent., so there was no need to carry out another cycle.

It is interesting to note that this extremely rapid convergence was obtained by balancing only, without making any attempt to correct for future operations. After working a few examples of any particular type of problem it becomes easy to estimate the probable

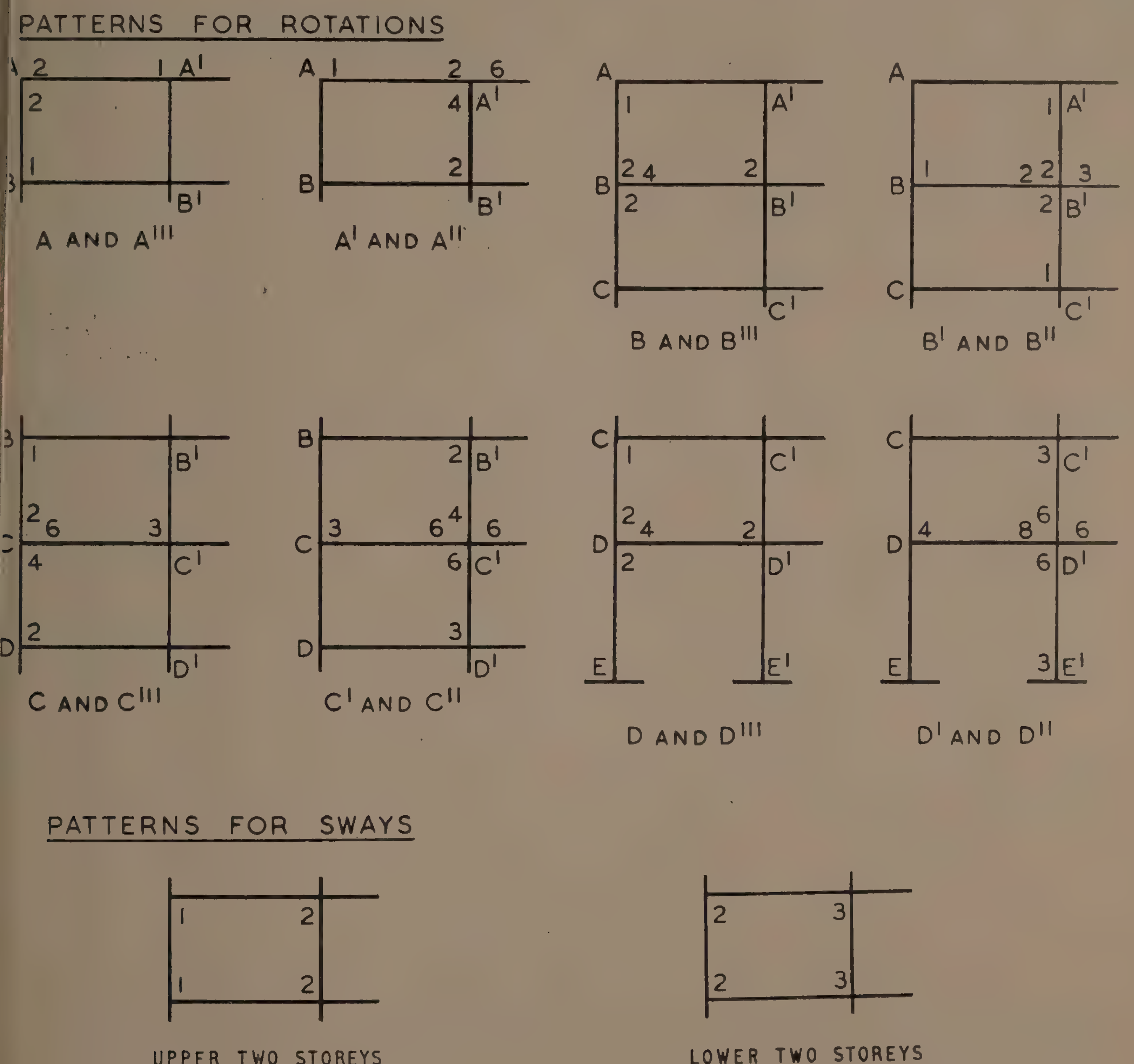


Fig. 14

pattern 2 of Fig. 15 (b) there are seen to be carry-overs of $10 \times .1 = 1.0$ to the beam at A, $10 \times -.1 = -1.0$ to the column at A and $10 \times -.1 = -1.0$ to the upper column at B. The balancing and carry-overs are shown in lines 1 and 2 of Table 2.

A was next balanced as indicated in line 3. The carry-overs were obtained from pattern 1 of Fig. 15 (b) and set down in line 4. In this way each joint of the frame was balanced, the order being A′, A, B′, B, C′, C, D′, D.

amount of over or under balancing required and possibly only one or two joints will need a second cycle to bring the largest error within 5 per cent.

4 (c). The Relaxation method is very similar, but instead of considering the moments in individual members the total moment at each joint is considered. In pattern 1, Fig. 15 (b) for instance the total moment at A′ is zero. Thus the operation of balancing at A will give rise to only two carry-overs, —.285 to B′, and

COMBINED "NO SHEAR" PATTERNS

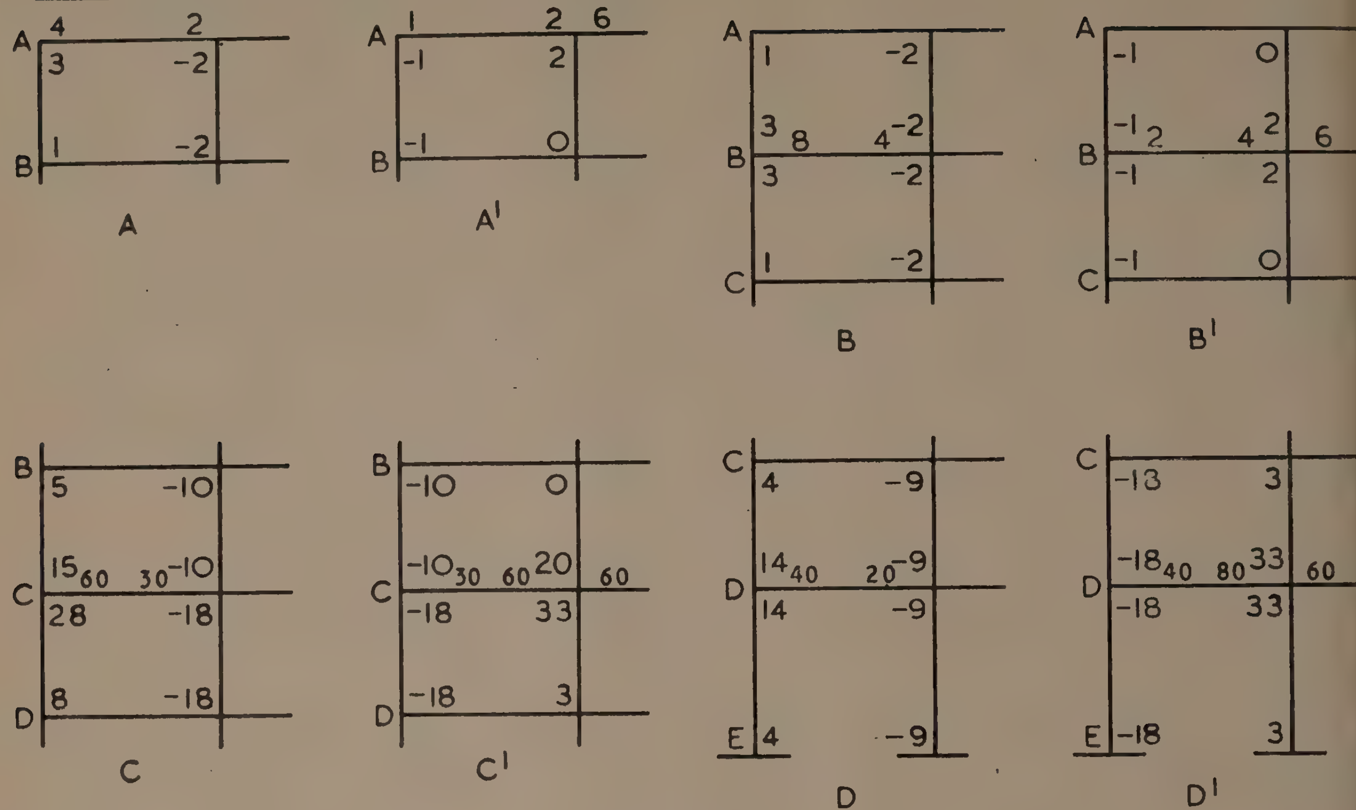


Fig. 15 (a)

"NO SHEAR" PATTERNS EXPRESSED IN DECIMALS

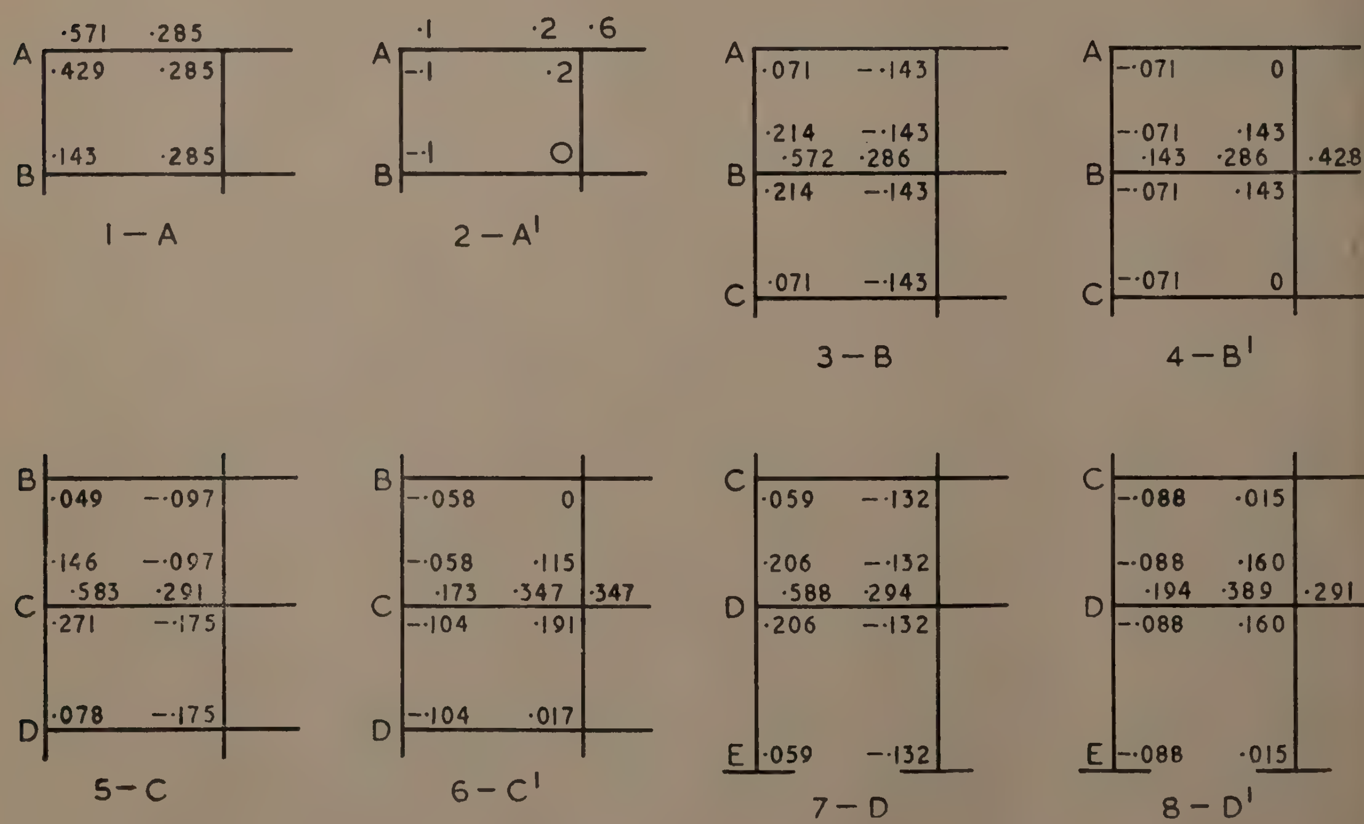


Fig. 15 (b)

TABLE 2

Bal'g Moment	E	D				C				B				A		
		86.0				45.63 +6.40 = 52.03				20.29 + 2.48 = 22.77				5.00 + 1.68 = 6.68		
		L	B	U		L	B	U		L	B	U		L	B	
SWAY M.s C.O. A′ BAL. A	—54	—54	0	—30		—30	0	—15		—15	0	—5 —1.00 +.72		—5 —1.00 +2.15	0 +1.00 +2.86	
C.O. B′ BAL. B								—2.96 +1.45		—2.96 +4.35	+5.92 +11.59	—2.96 +4.35		—2.96 +1.45		
C.O. C′ BAL. C				—8.1 +3.52		—8.1 +12.37	+13.5 +26.61	—4.52 +6.65		—4.52 +2.22						
C.O. D′ BAL. D	—11.6 +5.1	—11.6 +17.7	+25.8 +50.6	—11.6 +17.7		—11.6 +5.1										
C.O. C′ BAL. C				—.89 +.51		—.89 +1.73	+1.49 +3.74	—.50 +.93		—.50 +.32						
C.O. D′ B′ BAL. B	+.12	+.12	—.26	+.12		+.12		—.36 +.18		—.36 +.53	+.72 +1.42	—.36 +.53		—.36 +.18		
C.O. A′ BAL. A												—.32 +.24		—.32 +.72	+.32 +.96	
Final Moment	—60.38	—47.78	+76.14	—28.74		—31.36	+45.34	—14.13		—15.92	+19.65	—3.80		—5.14	+5.14	

L — Lower column at a joint.
U — Upper column at a joint.
B — Outer span beam.
B′ — Inner span beam.

Bal'g Moment	E′	D′				C′				B′				A′		
		132.6 —1.37 = 131.2				77.8 + 8.59 = 86.39				41.43 + 5.06 = 46.49				10.00 + 3.25 = 13.25		
		L	B	U	B′	L	B	U	B′	L	B	U	B′	L	B	B′
SWAY M.s 1 BAL. A′	—81	—81	0	—45	0	—45	0	—30	0	—30	0	—10	0	—10 +2.00	0 +2.00	0 +6.00
4 C.O. A 5 BAL. B′										+5.92	+11.84	—1.43 +5.92	+17.75	—1.43	+1.43	
8 C.O. B 9 BAL. C′				+1.35		+14.95	+27.00	—2.90 +8.95	+27.00	—2.90	+5.80	—2.90		—2.90		
12 C.O. C 13 BAL. D′	+1.9	+21.2	+51.6	—7.95 +21.2	+38.6	—7.95 +1.9	+13.30	—4.43		—4.43						
16 C.O. D 17 BAL. C′	—11.4	—11.4	+25.3	—11.4 +.14		—11.4 +1.64	+2.98	+.99	+2.98							
20 C.O. C 21 BAL. D′B′	—.02	—.22	—.53	—1.14 —.22	—.40	—1.14 —.02	+1.89	—.63		—.63 +.72	+1.45	+.72	+2.17			
24 C.O. B 25 BAL. A′								—.35		—.35	+.71	—.35		—.35 +.65	+.65	+1.95
28 C.O. A												—.48		—.48	+.48	
Final Moment	—90.52	—71.42	+76.37	—43.02	+38.20	—47.02	+45.17	—28.37	+29.98	—31.67	+19.80	—8.52	+19.92	—12.51	+4.56	+7.95

The above moments are for $P = 7.5$, therefore each should be multiplied by $\frac{P}{7.5}$ to obtain the bending moments for the load case given.

+.143 to B. These are indicated by arrows on a sketch of the structure Fig. 16, together with all other carry-overs derived from the other patterns of Fig. 15 (b). The sway moments of Fig. 15 (b) were also noted.

For comparison with the Hardy Cross method of accounting the order of balancing was made the same. At A′ for instance the out-of-balance moment was —10.00, the balancing moment required was +10.00, giving a carry-over of 10.00 × —.1 = —1.00 at B. At each joint the amount of balancing moment applied and the residual out-of-balance moment are retained.

To obtain the bending moments in the members after the balancing is complete a summation must be carried out. For instance, in this example, 6.68 times pattern 1 of Fig. 15 (b) plus 13.26 times pattern 2, plus 22.76 times pattern 3, etc., must be added to the sway pattern of Fig. 15 (b). Since Table 2 illustrates this kind of procedure a separate table has not been prepared.

It is of interest that the labour involved in the solution of problems by this method is directly proportional to their size. In most other methods, if the number of storeys is doubled the solution is much more than twice as tedious.

Example 6 (Fig. 17)

This example is chosen to illustrate the solution of frames with inclined members. The problem is solved by obtaining patterns for relatively simple load cases and building these up by inspection to the required solution.

In general it would appear that nine constraints are needed to hold the structure in position against any loading ; horizontal and vertical forces and a moment

at each joint B, C and D. Making the assumption that the members are inextensible, however, reduces this number to five, because the stanchions prevent vertical movement of B and D and the position of C is now dependent on the positions of B and D.

A moment constraint must be provided at each joint B, C and D to prevent rotations, but any two of the remaining constraints, horizontal forces at B, C and D, and vertical force at C, are sufficient to prevent displacement of the linkage ABCDE. Whilst the choice is entirely open to the operator, it is obviously best in this problem to choose the two most symmetrical constraints, namely, horizontal forces at B and D.

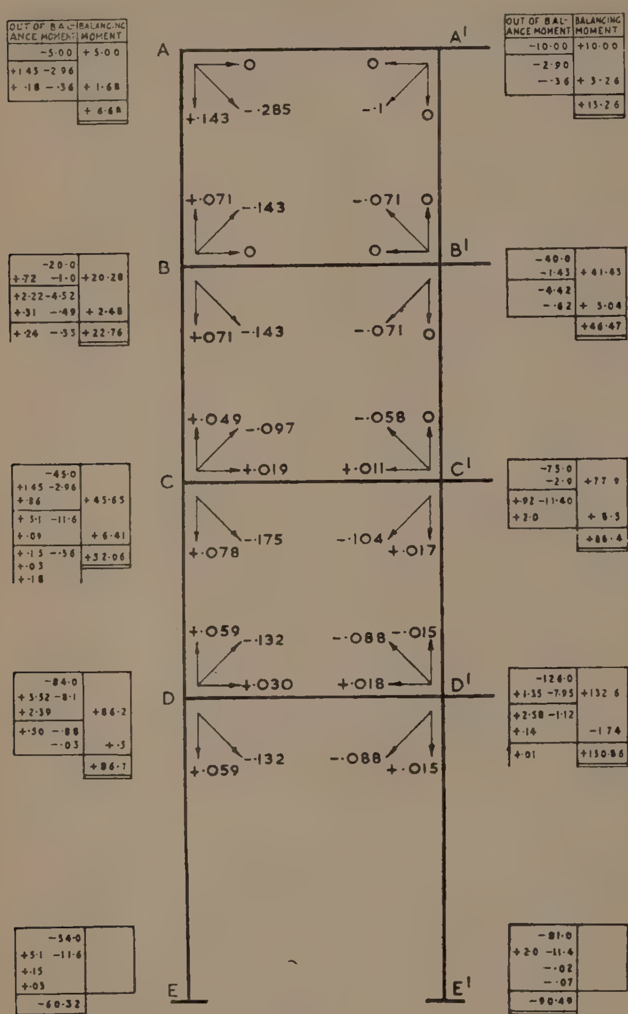


Fig. 16

A demonstration that horizontal forces at B and D only are sufficient to maintain equilibrium is apparent. If B and D are moved equal distances to the right, C moves to the right horizontally. If B and D are moved equal distances inwards, C moves vertically without horizontal displacement. Therefore, any combination of horizontal and vertical constraints at C can be provided by horizontal constraints at B and D, together with the vertical force provided by the stanchions.

Whatever choice is made, all forces appearing elsewhere must be carried back through the members to the constraints chosen.

1. *Fixed End Constraints*

$M_{CB} = -M_{BC} = M_{DC} = -M_{CD} = 8.33$ tons ft.

For this fixed end case a vertical reaction at C of five tons is required. Resolving the forces in the rafters, this is seen to require at B and D inward horizontal forces each of value five tons, and vertical forces which it is not necessary to calculate since they are provided by the stanchions.

2. *Calculation of Operations*

Since a horizontal force at C results in equal horizontal forces at B and D the value of the horizontal constraint at B (H_B) for any operation is given by :—

$$H_B = -\frac{M_{AB}+M_{BA}}{20} + \frac{M_{BC}+M_{CB}}{10} + \tfrac{1}{2}H_C$$

$$= -\frac{M_{AB}+M_{BA}}{20} + \frac{M_{BC}+M_{CB}}{10.2} - \frac{M_{CD}+M_{DC}}{10.2}$$

$$= \frac{1}{20}\left(-M_{AB}-M_{BA}+M_{BC}+M_{CB}-M_{CD}-M_{DC}\right)$$

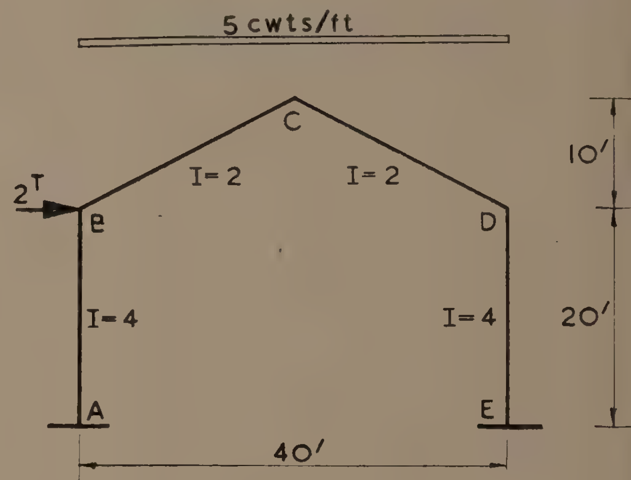


Fig. 17

WILLIOT DIAGRAM FOR DEFLECTION OF C FOR SYMMETRICAL SWAY OF B AND D

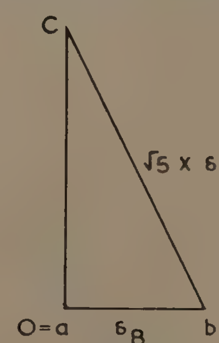


Fig. 18

Similarly

$$H_C = \frac{1}{20}\left(-M_{DE}-M_{ED}+M_{DC}+M_{CD}-M_{BC}-M_{CB}\right)$$

For sways $M_{BC} = M_{BA} \times \frac{2}{500} \times \frac{(20)^2}{4} \times \sqrt{5} =$ $0.894\ M_{BA}$

The five possible operations are listed at the head of 'able 3. Again there is a choice, since for instance the ways might have been chosen as the sway of B with D xed, and the sway of D with B fixed. These particular perations, however, were chosen for the same reason s the force constraints, to keep to symmetrical cases as ar as possible.

TABLE 3

	Operation	A		B		C		D	E	H_B	H_D
1	Rotation of B	·345	.69	.31	.155					—.0285	—.0233
2	,, ,, C			1	2	2	1			—	—
3	,, ,, D					.155	.31	.69	.345	—.0233	—.0285
4	Sway B→ D→	—1	—1					—1	—1	+.100	+.100
5	,, B→ D←	—1	—1	.894	.894	—.894	—.894	1	1	+.2788	—.2788
	Symmetrical Cases										
6	(1) — (3)	·345	.69	.31	.155	—.155	—.31	—.69	—.345	—.0052	+.0052
7	(6) + (5) × 9.43	—9.08	—8.74	8.74	8.59	—8.59	—8.74	8.74	9.08	+2.635	—2.635
8	(5) + (6) × 53.6	17.6	35.98	17.51	9.21	—9.21	—17.51	—35.98	—17.6	0	0
	Anti-Symmetrical Case										
9	(1) + (3) —.0775 × (2)	·345	.69	.233	0	0	.233	.69	.345	—.0517	—.0517
10	(9) — ·923 × (4)	—.578	—.233	.233	0	0	.233	—.233	—.578	.0406	.0406
	For Loading given Tons Feet Units										
	Original F.E. Constraints			—8.33	+8.33	—8.33	+8.33			+5.00	—5.00
	(7) $\times \frac{4.00}{2.635}$	+13.79	+13.30	—13.30	—13.09	+13.09	+13.30	—13.30	—13.79	—4.00	+4.00
	(8) $\times \frac{8.33}{53.49}$	+2.74	+5.60	+2.73	+1.43	—1.43	—2.73	—5.60	—2.70	0	0
	(10) $\times \frac{1.00}{.0406}$	—14.26	—5.74	+5.74	0	0	+5.74	—5.74	—14.26	+1.00	+1.00
	Final Moments	+2.27	+13.16	—13.16	—3.33	+3.33	+24.64	—24.64	—30.79	+2.00	0

. *Block Operations*

The solution to this problem can be built up when the lock operations are obtained for the following cases :—

(a) Equal and opposite moments at B and D.

(b) Equal and opposite forces at B and D.

(c) Equal and like forces at B and D.

The first two are obtained by combining operations ı), (3) and (5) of Table 3, since these are the only ymmetrical operations. Furthermore, any rotation of is accompanied by an equal and opposite rotation f D, so only two operations need to be considered, ı)—(3) and (5). These two are then combined to ive the required operations.

The anti-symmetrical case (c) is obtained from perations (1), (2), (3) and (4). In this case B and D otate the same amount and there are no moment onstraints. Therefore, (1) + (3) is balanced at C by he correct amount of (2) and the resulting out-of-alance moment at B and D is wiped out by (4).

4. The three block operations so obtained are used in the normal way of symmetrical and anti-symmetrical components to relax the fixed end constraints.

This paper is part of a thesis prepared in the Department of Building, College of Technology, Manchester, Head of Department : W. B. McKay, M.Sc.Tech., M.I.Struct.E. The paper was prepared under the supervision of W. Merchant, M.A., S.M., A.M.I.Struct.E., Reader in Applied Mechanics to whom the author is indebted for criticism and advice.

References

[1] " Elastic Stresses in Structures." E. S. Andrews. Scott Greenwood & Son. London. 1919.

[2] " Statically Indeterminate Stresses." Parcel and Maney. J. Wiley & Sons. New York. 1926.

[3] " The Stress Analysis of Continuous Frames." E. H. Bateman. THE STRUCTURAL ENGINEER. Vol. XIV. 1936. P. 398, 471.

[4] " Stress Calculation for a Radially Braced Polygonal Ring." R. V. Southwell and J. B. B. Owen. PHIL. MAG. Ser. 7. Vol. XX. 1935. P. 1073.

[5] " Analysis of Continuous Frames by Distributing Fixed End Moments." Hardy Cross. Amer. Soc. Civ. Eng. Proc. 1930. 56 (5), 919—28.

[6] " Relaxation Methods in Engineering Science." R. V. Southwell. Oxford University Press. 1940.

Structural Engineering and the Universities*

By Professor A. G. Pugsley, O.B.E., D.Sc.(Eng.), M.I.C.E., M.I.Struct.E.

A great deal has been said in recent years about science and technology ; the alleged inadequacy of the latter in this country and the place it should take in the educational world. I do not propose to discuss these questions, though obviously my subject bears upon them. I do not, in fact, propose to use the word " technology " at all ; it smacks too much of applied science and of industry to be a happy description of structural engineering.

I propose instead to try to do, in our own field, what I wish had been done before, and in others, by some of those who have already discussed the above questions : to survey objectively, though on this occasion necessarily in brief, the relationships that have grown up between the universities and practising structural engineers, and the contributions of one to the other. And if I look at all into the future, it will only be to suggest or point to small changes around the corner, rather than leap to ill-supported conclusions that seem to call for radical alterations in our educational system.

We tend to regard the link between universities and engineering as something that has developed largely in our own lifetime and that only started with the foundation of university engineering laboratories, the earliest of which in England date from about 1880. But this is by no means so, and particularly in our own subject ; long prior to the emergence of engineering laboratories, and even to the foundation of professorial chairs in engineering, university men interested in applications of mathematics, and of natural philosophy generally, played a part with the structural engineers of their time. Thus we find Professor Playfair, a mathematician of Edinburgh, advising a Select Committee in 1800 on Telford's proposal for a cast-iron arch of 600 ft. span across the Thames. At about the same time, one of our earliest empirical strut formulæ was devised by Eaton Hodgkinson to fit the results of tests on cast-iron columns—he was a Cambridge mathematician who later became Professor of Engineering at University College, London. The first general textbook on our subject—"The mechanical principles of engineering and architecture "—was written in 1843 by H. Moseley, sometime Canon of Bristol and Lecturer in Mathematics at King's College, London. Bow's notation for the application of graphic statics to bridge and roof frameworks also came from a lecturer in mathematics, in this case at Edinburgh.

In nearly all such early instances, the common interest that formed the bridge between university and the profession was an interest in statics—at once of fundamental interest to early applied mathematicians and basic to all structural problems. And although most mathematicians, at least during recent decades, have grown less interested in statics, and advances in the subject have tended to wait upon engineers, it still forms a point of contact. A valuable bridge in our own times between structural engineers and mathematicians has been the theory of elasticity, itself a product of statics combined with a study of linear deformation ; and University mathematicians, including many living to-day, have played a lively part in the development and engineering applications of this theory. Thus contact between University men and the profession was so natural that it occurred long before the foundation of chairs of engineering, and still is not canalised by such chairs.

I have mentioned " chairs of engineering " rather than " chairs of structural engineering " advisedly ; so far as I know there has never been a University chair of structural engineering in this country, and yet we are all well aware there always have been University professors and lecturers in engineering faculties who were interested in structures. At the moment the universities of this country provide some 10 chairs of civil engineering and a further 10 either jointly in civil and mechanical engineering, or more generally, in engineering ; and of these 20 chairs about half are occupied by men whose primary interest is in structures. Such a large proportion may not always have obtained ; it is certain that in earlier days such specialisation of interests was neither possible nor common ; but it is, I think, fair to say that the staffs of university engineering faculties have, so far as civil engineering subjects are concerned, commonly shown a bias towards structures. As a result, in spite of the absence of specific reference to structures in the titles of university chairs and departments, I do not think that either the Institution of Civil Engineers or the newer and more specialist Institution of Structural Engineers, have ever had real occasion to feel that their professions were ill-served by lack of interest in the universities. Indeed, the position has on occasions been a reverse one ; the profession has sometimes looked askance at the growing number of young graduates, academically sound, may be, but strangely lacking in practical knowledge and experience, and sometimes even in any realisation of the value of experience. And it is only in the last 20 years that the universities have become so common a channel for entry into the profession.

It is perhaps as well to remember that the position might have developed otherwise. In the early nineteenth century the growing body of civil engineers interested in structures—particularly bridge structures at that time—sometimes looked upon their duty to the next generation in terms of quite other possibilities. They had grown accustomed, during and following the Napoleonic Wars, to the presence of a military corps of engineers with cognate interests ; they were well aware of the older and then more famous Corps des Ponts et Chaussées of France ; and they had at least a fragmentary knowledge of the ancient Monastic order of Bridge Builders that once operated throughout Europe. So that even in 1870, we find the man who back in 1841 had become the first professor of engineering in England, and probably also in the world, we find Charles Vignoles in his presidential address to the Institution of Civil Engineers drawing keen attention to the power and value of the French system. To those of us who have come in contact with that excellent representative in this country of this type of development, the Royal Corps of Naval Constructors, the effectiveness and power of such an approach are well known. But the demerits of such inbreeding are clear to many also, not least to the senior members of the Corps itself, and here the forging of a link with the universities is now regarded as one of the ways of improvement.

**Chairman's address to the Western Counties Branch of the Institution of Structural Engineers given at Bristol on October 5th, 1951*

In mentioning thus briefly the early history of the interplay between structural engineers and University men, I have really touched upon a number of the ways in which this can occur. Let us look at these a little more closely and in more modern terms. In the first place, just as the early mathematicians developed structural theories and helped the engineers to apply them, so it is still customary for University engineers and mathematicians to conduct research—experimental now as well as theoretical—and again, by direct contact as consultants or advisers, to help structural engineers to utilise their results. And just as Hodgkinson's work on struts referred to the needs of his time—needs he had learnt from such men as Stephenson and Fairbairn—so now University research in engineering is commonly related, albeit not always obviously or closely, to the needs of our time. The steel structures researches at Cambridge, the reinforced concrete work at Leeds, and light alloy work at Bristol, are instances of this ; and in all these instances, the University staffs concerned enjoy the closest support and collaboration from the practising members of our Institution.

But it would be unwise for the Universities, and unhealthy for the art of structural engineering, if all the researches at universities were related directly to current needs. It is not only nice, in the æsthetic sense, to see work on the fracture of glass at Cambridge, on the creep of struts at London, and perhaps on probability and safety at Bristol; it is vital to the intellectual wellbeing of the Universities and in the long run to the fundamentals of our art. Ask us to work on your problems, to collaborate with your research associations and with Government establishments, but let us indulge a little our own personal curiosities and so keep our souls.

From contact on the research and consulting level, let us turn to the other end of the story—the movement of University graduates into the structural engineering profession. For a long time after the introduction of engineering courses into University work the numbers of students were so small that the profession scarcely recognised their existence. Thus, in his Presidential Address to the Institution of Civil Engineers in 1866, Sir John Fowler is still able to say quite naturally : " We of the passing generation have had to acquire our professional knowledge as best we could, often not until it was wanted for immediate use, generally in haste and precariously, and merely to fulfil the purpose of the hour "—a position which even now some of us recognise —and to refer to the rare case of a would-be engineer going up to Oxford or Cambridge as entering upon an interregnum for the benefit of his general culture to the peril of his continued interest in engineering.

By 1892, however, the impact of the new engineering graduates was being felt, with results which are still sometimes only too familiar. In that year, we have Sir Benjamin Baker, Sir John's younger partner, enjoying the following story so much that he introduced it, willy-nilly, into a discussion on graving-docks !

" Another case was from the Antipodes, and that was really so good that he would like to read out of a Blue Book, containing a report of the proceedings of a Royal Commission, one or two questions and answers in illustration of the point he was referring to. ' Q. How long have you been an Associate Member of the Institution of Civil Engineers ? A. About two years. Would you like to know why it is such a short time ? Q. I did not ask you. A. Well, I would like to qualify the answer. The reason I did not become an Associate Member before was that when I contemplated coming to Australia, I was certainly not aware that they thought so much of the fact that a man was a member of the Institution ; but when I arrived here, I found, to my surprise, that they thought more of that Institution, in which the greatest number of duffers that I know in the profession are congregated, than of the University degree of Master of Engineering, which I hold, one of the highest in the United Kingdom. Learning that such was the case I became a member, as I say, two years ago. As a matter of fact, I could have been a member at least ten years previously. Q. But you are not a member now ? A. I am a corporate member now.' That gentleman," said Sir Benjamin, " was labouring under the delusion that the degree of ' Master of Engineering ' in some strange way constituted a man an engineer. . . . A case like this should serve as a warning to students against imagining that if they had taken a degree in engineering, or had left a University with the highest credentials, they were therefore necessarily engineers."

Since those times, civil engineers, in particular, have progressively encouraged young aspirants to their profession to proceed to University departments of engineering, so that this step has become a routine with them. In our own Institution the same process has never applied so generally. I was aware that the origins of this Institution were naturally such that in 1908, the year of its foundation, the proportion of graduates to be found among its members was small. But I was a little surprised the other day when, for the purposes of this address, I dipped into sample pages of the current year book and found that, among Members and Associate Members alike, only about 20 per cent. appear to have University origins. One can see reasons for this, particularly of course the youth of our Institution and the tendency to an overshadowing by the senior Institution, but I think there is cause here for thought by both Institutions as well as by the Universities. Though so far as the last are concerned, Mr. President, I myself was able to derive some comfort from the fact that the general figure of 20 per cent. rises, in the case of our Council, to some 50 per cent. ! The corresponding figures for the Institution of Civil Engineers are roughly 55 per cent. and 65 per cent. All these figures are probably slowly increasing.

We have for long been accustomed to looking to University men as the main source of books and general monographs on our subject, as well as a fruitful source of research papers. In the nineteenth century our grandfathers studied Rankine and Moseley, and our fathers looked to Unwin and Claxton Fidler ; we ourselves may have been brought up on Ewing and Morley. But many of to-day's undergraduates, through no fault of their own, tend to feed on foreign fare ; of that more in a moment.

This dependence of each generation upon the works of its teachers is of course a very natural one and one that most of us, in our own lives, have come to value. It provides for each of us a standard from which to view engineering thought, before and after ; and in the past, at least, the Universities have provided just the conditions that go to the making of good text-books—contact with able students, freedom from routine duties, and refreshing leisure. Some of us may have thought these conditions were overdone, that just as good books would have been forthcoming for each generation if University life had been harder in the " nose-to-the-grindstone " sense, but before we pronounce on this let us observe what has been happening in our own lifetime.

The natural process I have outlined was interrupted by the 1914-18 War, but fortunately and, as it then appeared, necessarily, this was followed by a spate of good books, such as those by Southwell and Pippard.

A strange lull has intervened ever since ; the 1939-45 War has been followed by no new spate, and to-days' students are turning more and more to America for their up-to-date books. I am not referring here to routine productions or to the popular works to be seen on station bookstalls—which have become all too common—I am referring to serious text-books by recognised leaders, particularly in the Universities.

This change is, in my view, a serious one that should if possible be combated. I suspect it springs partly from the heavy advisory duties that, in the last twenty years, circumstances and the Government have seen fit to put on the shoulders of its University men, and as much on engineers as any. This is a trend, of course, that plagues most small countries. In the long run, the fate towards which we may be moving in this book world is the fate of the second-rate nation. Ways of avoiding this must be found, and both thought and action in the matter are already overdue.

We are living in an era of congresses, conferences, and summer schools, and in these our own profession and the Universities have somewhat gaily played their part. But the more one attends such meetings, the more one regrets the machine-made discussions, hedged about by time and printing limitations, and the more one values the incidental private conversations that have been squeezed in during the meetings in spite of the organised programmes. The more one feels too, how much better it would have been if this particular congress were held once in ten years instead of biennially, and the like. Leisure for thought and mental refreshment seems again to be the key, a key which the atmosphere of a University traditionally provided and must strive to retain. If our structural engineering profession can avoid the prevailing too-frequent rush to conferences, and can at the same time find and maintain leisurely contacts of an informal kind, then it will have achieved something of lasting import that Universities should applaud and help. This is one of the pleasures and advantages of Branch meetings, at least on less formal occasions than the present.

Our problem here, of course, as in our discussion on text-books, reflects a trend that is not a structural engineering matter or even a national one, but at least a European one. And in defending leisure I am in a sense opposing the tide, beneath which no doubt I myself have been partially submerged. But I am nevertheless unrepentant in believing that Universities should in this matter provide something like the cloisters of the ecclesiastical structures from which they derive, not only for their regular brethren but also for the refreshment of lay visitors ; cloisters in which a knowledge and understanding of the history of our art may grow and an opportunity to dream of its future may exist. From such meetings of practical and academic culture much has come in the past and I hope much will come in the future.

Comparative Tests on Various Types of Bars as Reinforcement of Concrete Beams

Discussion on Dr. K. Hajnal-Konyi's Paper*

Dr. Hajnal-Konyi said that, in view of the length of his original paper, he had had to omit from the published paper his introduction, which formed the background for the tests described. Therefore, he would read the introduction in order to put the matter in its proper perspective. He had wanted to avoid omitting any of the technical information because he wanted readers of the paper to arrive at their own conclusions.

He illustrated by slides some strain measurements as well as another series of tests on beams reinforced with untensioned 12g wires of an ultimate strength of 120 t/sq. in.

Dr. P. W. Abeles (Member) congratulated the author on the tests in which he had been able to fracture non-prestressed wires having a strength of 120 tons per sq. in., which had previously been considered to be impossible. Dr. Hajnal-Konyi had twice been able to prove something which Dr. Abeles had thought would be the case but which he had no opportunity of testing. Eight years ago Dr. Hajnal-Konyi had been able to prove that the tensile strength of steel well embedded in concrete was higher than that in the air. Dr. Abeles expected this, but he had not thought it would be possible to prove it in so simple a way. In the present paper Dr. Hajnal-Konyi had again proved a very important point, i.e., that cold drawn wire of a very high strength used as ordinary reinforcement may fracture at failure of the beam.

Dr. Abeles said that he had had the opportunity of seeing the test just before, and at, failure of the beam No. 20H, as described by the author, and he exhibited a load-deflection diagram in which had been plotted the test results with regard to beam 20H together with those of some other beams (4H, 5H, 8H and 11H). In addition to the load the bending moment also was plotted and the concrete stress f_1, calculated for the bottom fibre of beam 20H for a homogeneous section.

It was seen that cracks developed in beam No. 20H at a concrete tensile stress of somewhat below 600 lb. per sq. in. If the wires, or some of them, had been tensioned, a much higher load would have been reached at visible cracking. For a high strength concrete of a cube strength of 6,000 to 7,000 lb. per sq. in., a tensile bending stress $f_1 = 1{,}000$ lb. per sq. in. would apply, but in the present case $f_1 = 950$ lb. per sq. in. was taken into account, as shown in the graph, in view of the lower strength of 5,400 lb. per sq. in. of the concrete. If half the wires were tensioned, an effective prestress of 470 lb. per sq. in. would be attained at the bottom fibre, resulting in a cracking load corresponding to a stress of $950 + 470 = 1{,}420$ lb. per sq. in., whilst with full pre-

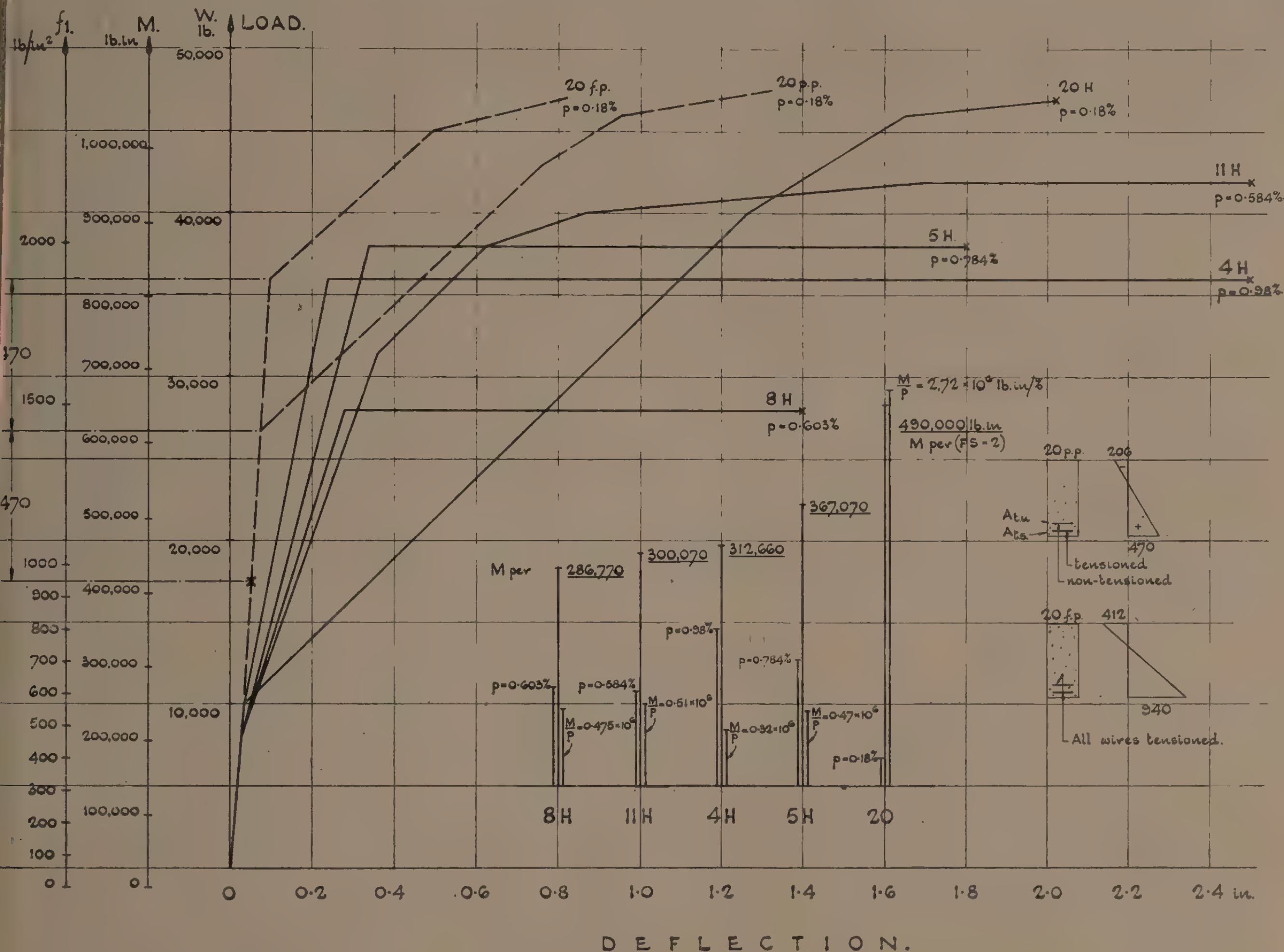
**Presented at a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, May 17th, 1951, Dr. S. B. Hamilton, M.Sc., M.I.Struct.E., A.M.I.C.E. (Vice-President), in the Chair. Published in* THE STRUCTURAL ENGINEER, *Vol.* XXIX, *No.* 5, *p.* 133.

ressing, i.e., at tensioning of all wires, the cracking ad would correspond to a stress of 1,420 + 470 = ,890 lb. per sq. in. The loading diagrams were indiated in the graph representing these two cases of artial prestressing (No. 20 p.p.) and of full prearessing (No. 20 f.p.).

The beam 20H was very interesting from a general oint of view, but would be unsuitable for practical use, s Dr. Hajnal-Konyi has already pointed out, because f the large deflection ; but a beam in which a part of

In conclusion, Dr. Abeles said that the very important investigations made by Dr. Hajnal-Konyi had shown the importance of efficient bond when high strength steel was used, and this applied also to prestressed concrete.

Mr. A. Hiley, A.M.I.C.E. (Member), expressed his great admiration for Dr. Hajnal-Konyi's paper and his work, and said he had elaborated the conception utilised in deformed bars to advantage and had revealed the



he steel were tensioned as, for example, 20 p.p., would epresent a great improvement to any of the beams hown, in view of its high ultimate load and high degree of safety against cracking.

In the graph there were also plotted for each example he percentage, the permissible bending moment and values $\frac{M}{p}$, representing the ratio of the permissible ending moment and the percentage ; this indicated to what extent the steel was utilised. The permissible ending moments of the various beams were given in Dr. Hajnal-Konyi's paper and that of a prestressed beam 20, be it fully or partially prestressed, was taken s half the bending moment at failure, ensuring a factor of safety $FS = 2$. It was seen that beam 20p combined he greatest permissible bending moment with the lowest ercentage ; consequently the value $\frac{M}{p}$ became a maxinum.

possibilities applying to the different strengths of concrete which were available at the present day.

The author had referred to the construction of certain work in reinforced concrete during the First World War, in which deformed bars were used. About 1915-16 Mr. Hiley was concerned with shipbuilding, and he was deputed to organise and to manage a shipyard specifically for the construction of ferro-concrete cargo boats and barges to effect an economy in weight of steel used at Barrow-in-Furness. He had had to consider at the design stage what should be done, in view of the nature of the estimated bending stresses, when a reinforced concrete vessel in a seaway was loaded to the deep draught, carrying 1,200 tons of deadweight. It had been necessary to consider whether to use ordinary round steel bars or deformed bars to cope with the prolonged reversals of stress anticipated, and he had chosen the spiral-bond bar as best suited to avoid slip reliably. Those bars were thereupon adopted for the sea-going ship at a design stress of 9 tons/sq. in., which corresponded to the higher elastic limit of, he believed,

60,000 lb./sq. in. in the deformed cold twisted bar. Its shape was approximately of hexagonal section, in which three of the sides were straight and the others curved. For bars of $1\frac{1}{8}$ in. diameter the degree of twist given was one twist in 12 in. In such large sizes some difficulty was met in bending the bars to form anchorages in the floors where necessary. The vessels being comparatively heavy had to endure somewhat high shear stresses in the launching condition, whilst in the hogging condition in a seaway the height of the waves being assumed one-twentieth of the length of the vessel, reversals of stress were repeated every five or six seconds. He believed the decision to employ the deformed bars in the construction of large ships was wise, as in the severe conditions stated, bond failures had remained unknown.

Mr. Hiley recalled having voyaged in one of the ships through a storm when steaming in the light condition. The ship had machinery at the after end and, therefore, trimmed considerably by the stern. When the sea calmed sufficiently he had observed on the fo'c'sle continuous deflections of $\frac{1}{4}$ in. amplitude the frequency of the vibrations taking effect there being 240 per minute. Such continued shaking as occasionally encountered in certain conditions was a very clear indication of the importance of safely dealing with bond stresses which the steel bars had to take without slip, and he considered it fortunate that they had a margin over ordinary round steel bars in the s.s. *Armistice* as completed in 1918.

Eventually this ship served very successfully in the coastal trade of West Africa ; she was laid up only a year or so ago, the cause being engines which had given out, not the concrete hull nor the reinforcement. That described an interesting but isolated example of successful results attending the use of spiral-bonding bars in ship construction.

There had been considerable difficulties to face in getting the special steel supplies regularly at the shipyard, and considerable trouble was also met in bending the steel of high elastic limit to the usual requirements ; bars had in many cases to be heated in the smith's fire. This work was naturally found rather difficult and slow compared with ordinary mild steel reinforcement, so that at Barrow and elsewhere in the construction of barges of 1,000 tons deadweight, the round bars in those cases were everywhere adopted to the normal British Standard specification, the manufacturing conditions as well as deliveries being so much easier to comply with in mild steel.

He believed the use of spiral bars had to drop out of fashion chiefly on that account. But he emphasised that if vibration was an important factor to contend with and if for some special reason ships were to be constructed of reinforced concrete in the future, the spiral bond or its equivalent might again be found very serviceable.

He did not know whether Dr. Hajnal-Konyi would go as far as to advocate the use of a corrosion-resisting steel containing copper for certain applications like those of shipbuilding, where twisted steel reinforcement may be selected to keep the steel weight to a minimum in floating structures.

Mr. T. C. Durley said that the author had rightly brought out the importance of being able to bend a high-tensile bar. It seemed to him that in the case of twin-twisted or Isteg bars, for any given radius of bend, we were bending a relatively much smaller size of bar than where a single high-tensile bar was adopted. Apart from the frictional effect, which was negligible, the moment of inertia of an Isteg bar was the same about one direction as about another. It was his experience that it was easier to bend the Isteg bar than any of the other cold-worked bars. He asked for the author's views on the matter.

Mr. O. Bondy, A.M.I.C.E. (Member), said the author had made it clear that he had referred to the various types of reinforcing bars from the point of view of their performance in reinforced concrete beams, and in his tests he had compared their performances. He had thrown out a challenge, not only to the designer, but especially to the steel-maker. He had referred to certain requirements which, if met, might help the designer in securing greater economy in the use of steel in reinforced concrete beams.

Mr. Bondy had a feeling that steel-makers would be in a better position to improve certain properties of steel as used for reinforcing bars if only they knew what were the designer's requirements and what demand was to be expected (say) per annum. A somewhat similar position had arisen in the development of certain qualities of steel for welded construction, and it was found that the steel-makers were only too keen to satisfy the demand if they knew where they stood.

The problem of corrosion resistance had been mentioned by speakers, and it had occurred to him that the question of weldability might also deserve a word. The weldability of reinforcing bars did affect the design of a number of major reinforced concrete structures in the past, and it might be expected that it would also affect design in the future. Mr. Bondy drew the attention of Dr. Hajnal-Konyi to a certain type of reinforcing bar which he had seen illustrated in a Swiss paper recently. Not only were high mechanical properties claimed for that type of bar—the ultimate tensile strength being 47 to 57 tons/sq. in., i.e., well above the 37 to 43 tons range of our B.S. 548—but in addition it was claimed that the bar was weldable. That was something new to him, and perhaps Dr. Hajnal-Konyi could say whether it was feasible that high tensile steels, produced by cold drawing or other types of cold work, could at the same time be made weldable, without losing their tensile and yield strength.

Dr. Hajnal-Konyi, replying to the discussion, first thanked the meeting for the kind reception accorded his paper. Although it was rather critical of present-day practice, nobody had contradicted his conclusions. He did not know whether or not he could assume from that fact that the meeting was in agreement with his conclusions.

Dealing with Dr. Abeles' contribution, he said he had found the comparison which Dr. Abeles had made on his graph was very interesting. It was suggested that the high tensile wire he had tested, as described in the lecture but not included in the paper, would be a practicable proposition if it were at least partially prestressed. Dr. Abeles' graph had shown a close approximation to the expected behaviour if half of the wires were prestressed ; and it would be a practicable proposition. But in the lecture he had pointed out that the tests he had made on high tensile wires had nothing to do with practical applications ; he had wanted to see the development of cracks and to show that steel could be broken in spite of its very high strength. The term $\frac{M}{p}$, which he had not seen before, seemed to be a

very valuable characteristic to use, not only for permissible moments, but for ultimate breaking moments.

He had had no experience of ships of reinforced concrete construction, but Mr. Hiley's remarks had made clear how useful it was to take advantage of cold-worked steel in shipbuilding. As to the difficulty about hooks, Dr. Hajnal-Konyi was of the opinion that cold-worked steel with a suitable surface pattern did not need any hooks, in contrast to plain round bars. That was now acknowledged in the new Code of the American Concrete Institute, where it was definitely stated that plain round bars had to be hooked, whereas deformed bars complying with the latest Specification of the A.S.T.M. need not be hooked. Nevertheless, he attached much importance to the cold bend test, since it was necessary that it should be possible to bend the steel to any required shape. The provision that cold-worked bars need not be hooked did not apply to all cold-worked bars; and he recalled Professor Ros' conclusion that it was necessary to provide hooks for Torsteel bars if the concrete strength were below a certain limit.

Replying to Mr. Durley, bars included in his series were bent through the radius as specified in B.S.S. 1144. That was done twice, in the laboratory and in an ordinary industrial bending machine. He did not find that the twin-twisted bar had any advantage in comparison with single twisted bars of equivalent size.

He was not very conversant with the question of the weldability of reinforcing bar steels, and there were others much more qualified than he to answer Mr. Bondy. So far as he was aware, cold-worked steels were not particularly suitable for welding, but apparently some people had welded them successfully and there were certain claims for cold-worked steels. The difficulty about welding cold-worked steels was that, if they were exposed to a high degree of heat, the value of the cold work was lost; therefore, there was a danger that, if cold-worked steel were welded, the strength at the neighbourhood of the weld would revert to the strength of the parent metal. It was possible, of course, that the high temperature was not applied for so long as to have a detrimental effect on the strength. But he had not made any tests in that connection and he could only repeat what he had heard.

Commenting on Mr. Bondy's reference to the point of view of the steel-makers, he said he looked upon the question of cold-worked steel as being very important under present circumstances, in view of the steel shortage. It we used cold-worked steel, the saving in the quantity of steel used over the country as a whole would be very considerable; and he claimed at the same time that the quality of the structures in which it was used would improve. But the problem was not very simple. It seemed that there were three conditions which must be fulfilled. First, we must have a Code which allowed the use of high quality steel to its best advantage. At present that was excluded; but until we had such a Code, every attempt at improvement was doomed to failure. We knew the industry was nationalised; nevertheless, it was not a proposition to embark on the production of a new steel or quality of steel if no advantage was to be gained by its use. Therefore, the first step required was to increase the stress permitted by the Code. He had tried to prove that that would not mean a reduction of the factor of safety; on the contrary, with a suitable quality of steel it would mean that the factor of safety could be higher. He did not think engineers would be justified in asking for a higher factor of safety; but there was so much margin that higher stresses could be allowed and the factor of safety could be improved.

In that connection he referred to the latest Dutch specification, recently received, in which permissible stresses went up to 35,000 lb./sq. in. to enable the use of a new type of steel which was not yet available on the market.

The second requirement was to produce steel of improved quality.

The third, which was equally important, was for engineers to accept the development and to take advantage of the availability of better quality steels, realising that if the steel quality were improved, the structures would be better.

We had to proceed step-by-step, and he believed there was an urgent requirement for a drastic revision of the Code to enable further development on the one hand, and on the other hand to exclude bond failures by taking into account the hook length, which was not justified in his opinion, and by specifying the radius for the hook in the bar. By attention to such matters we could increase the safety of structures.

Written Communication

Mr. J. Wiles (Associate-Member), expressing his appreciation of Dr. Hajnal-Konyi's very interesting lectures, said it was always reassuring to the designer to be able to compare practical test results with the theoretical values—especially when the tests substantiated the theory!

Referring to (4) in the author's suggestions for the amendment of the Code and B.S. 1144 at the end of the paper, Mr. Wiles invited him to amplify his statement: "Thus, at the 'economic' percentage, mild steel cannot be replaced by an equivalent area of high tensile steel without the addition of steel in compression. *This is not justified by experimental evidence.*"

Did that mean that in practice the neutral axis would not rise, or did it mean there was not a linear stress distribution over the cross-sectional area of the beam? If it were not linear, was it non-linear within the working load? Was it parabolic or rectangular?

The Author replied: The meaning of the statement referred to by Mr. Wiles was that the position of the neutral axis at failure did not depend on the percentage of reinforcement p and the modular ratio m as assumed by the standard method but on the factor $p\dfrac{t}{c_p}$ where t was the stress in the steel, c_p the prism strength of the concrete. As long as $\dfrac{pt}{c_p}$ remained constant, there was no change in the position of the neutral axis. This was confirmed by strain measurements on 11 beams. The stress distribution in the concrete at working loads was straight with a good approximation, but this did not apply to loads approaching the maximum. The stress distribution of the concrete at failure was not known and was probably very different in different qualities of concrete. However, Whitney's simplified assumption of a rectangular stress distribution allowed a close approximation of the lever arm at failure and thus the prediction of the maximum bending moment if the strength properties of the materials were known and p did not exceed a certain limit. This limit was far in excess of the so-called "economic" percentage as obtained by the standard method.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, November 22nd, 1951, at 5.55 p.m., Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership ?

Students

BAILEY, Edwin Roy, of Guildford, Surrey.
BARWIS, Edward Charles, of London.
BOSMAN, Edward Albert, of Durban, South Africa.
GOODE, Lawton Thomas, of London.
GRASSMANN, Brian Godfrey, of Johannesburg, South Africa.
HALLOWS, Peter Marcus, of Wirksworth, Derby.
HARRIS, Colin Frederick, of Umtali, Southern Rhodesia.
HIGSON, Martin, of Manchester.
HODGSON, William, of Paarl, South Africa.
HOLLEY, John Henry, of Much Dewchurch, Hereford.
KROL, Tadeusz, of London.
LENARTOWICZ, Witold, of London.
LEVTOV, Louis, of London.
LITHGOW, Ian Spence, of Clarkston, Renfrewshire.
MCHUGH, Patrick Thomas, of Hounslow, Middlesex.
MANKARZ, Otto Heinz, of Salisbury, Southern Rhodesia.
MATTOCKS, Ronald, of London.
MEAD, Peter Frank, of Exmouth, Devon.
MITCHELL, John Cyril, of Belfast, Northern Ireland.
MORCOM, Peter John, of Durban, South Africa.
ODEDAIRO, Ebenezer Olufunso, of London.
PEGLER, Michael Richard Holmes, of Johannesburg, South Africa.
POOLE, Liam Myles, of Durban, South Africa.
PRIMROSE, David James, of St. Helens, Lancs.
PROBERT, Leslie Anthony, of Nottingham.
TANDY, Robert Macmillan, of Germiston, South Africa.
THOMPSON, Peter Smithson, of Harrogate.
TONGE, Brian Yardley, of Bolton, Lancs.
WILKS, Leslie Robert, of London.
WINFIELD, Peter Frederick, of Middlesex.

Graduates

AAB, Frank Albert Werner, B.Sc.(Eng.), Rand, of Edenvale, Transvaal, South Africa.
APPLETON, Samuel Alan, of Liverpool.
ASCOUGH, Dudley William Allison, of Middlesbrough, Yorks.
BETTS, Anthony Charles George, B.Sc.(Eng.)(Hons.), London, of Newark, Notts.
BRANCHER, David Marshall, of London.
BUSHELL, Charles Joseph, of Boston, Lincs.
CAZALY, Laurence George, B.Sc.(Eng.), Bristol, of London.
CHATTERJEE, Shyamal Prosad, B.Sc.(Eng.), Patna, of London.
DUCKWORTH, Frederick John, of Crayford, Kent.
EVANS, Peter Robert, of Greenford, Middlesex.
FAYYAZ, Ali, B.Sc.(Civil), Punjab, of Dera Ghazi Khan, Pakistan.
GRAFF, Jacob Yehuda, B.Sc.(Eng.), Rand, of Johannesburg, South Africa.
GUPTA, Ramkrishna Shankarlal, B.E.(Civil), Bombay, of New Delhi, India.
HALL, Thomas Robert, B.Sc., Belfast, of London.
HENNY, Gerhardus Evert Jan, B.Sc.(Civil), Rand, of Johannesburg, South Africa.
HODGKINSON, Allan, M.Eng., Liverpool, of London.
HOLMES, Eric William, of Twickenham, Middlesex.
JINARAJAN, Govindan, B.Sc.(Civil), Travancore, of Leeds.
KADIANI, Fida Husein Chulamali, B.E.(Civil), Bombay, of London.
KLOPPER, Stephanus, of Dieprivier, South Africa.
LEWIS, David John Donald, of Pontardulais, Glam.
LOFTS, Roy Edgar, of Worthing, Sussex.
MAHFOUZ, Georges Elias, B.Sc.(Civil), Giza, of Cairo, Egypt.
MEADS, John Richard, of Walsall, Staffs.
MOTIVALA, Jijoo Framroze, B.E., Poona, of Bombay, India.
NANDI, Subir, B.E.(Civil), Calcutta, of London.
NOHR, Max, of London.
PARDOE, Maurice Geoffrey, of Middlesbrough, Yorks.
PEACOCK, John Desmond, B.Sc.(Eng.), London, of St. Albans, Herts.
PERCHARD, George Desmond, of Runcorn, Cheshire.
PINTO, Vitorino Antonio C. L., B.E.(Civil), Bombay, of Goa, India.
POOL, James Fraser, B.Sc.(Civil), Rand, of London.
SCAHILL, Gregory John, B.E., B.A., Sydney, of Lakemba, Australia.
SHVARTZ, David Itzhak, of Tel-Aviv, Israel.
SIMMONDS, Charles Arnold, M.A.(Cantab.), of Lytham St. Annes, Lancs.
SIVALINGAM, Peethamparam, B.Sc.(Eng.), London, of London.
STRONG, Gerald Albert James, A.M.I.Mun.E., of Bath, Somerset.
SUBBA RAO, Tippur Narayana Rao, B.E.(Civil), Mysore, of Bangalore, India.
TAYLOR, Gerald, of Reading, Berks.
TSAO HSIEN HWA, of Singapore.
TURNER, Geoffrey, of Scunthorpe, Lincs.
VASWANI, Harkrishin Pahlajrai, B.E.(Civil), Bombay, of Croydon, Surrey.
WATTS, Donald Ernest, of London.
WHALE, John Frederick, M.Sc., Auckland, of Auckland, New Zealand.
WOOD, Peter Buckley, B.Sc.(Tech.), Manchester, of Salford, Lancs.

Associate-Members

CUTLER, Kevin Mulqueen, of Wellington, New Zealand.
DAHM, Hans, of Cape Town, South Africa.
FALISZEWSKI, Stefan Wilhelm, of Montreal, Canada.
MINHAS, Mohammed Hussain, of Lahore, Pakistan.
RAFE, Abdur, of Lahore, Pakistan.
SAVONA, Joseph Sigismund Ethelwold, of Sliema, Malta.

Members

BUTTON, Harold, B.Sc.(Eng.), London, of Birmingham.
DOANIDES, Peter John, of Johannesburg, South Africa.
ROOKS, George Joseph, of Durban, South Africa.
SMITH, John Arthur Gordon, of Epsom Downs, Surrey.
SPYRA, Jan Jakub, A.M.I.C.E., of Manchester.

Transfers

Students to Graduates

ALLOTT, Ernest James, B.Sc.(Eng.), London, of Rotherham, Yorks.

ATTWOOD, William Enoch Edward, of Wynberg, South Africa.
BANERJEE, Hari Har, B.Sc., Calcutta, of Calcutta, India.
BROCKBANK, Edwin, of Stretford, Lancs.
DOYLE, William Sherwood, of Cape Town, South Africa.
GARNER, Eric Cyril, of Prestwich, Lancs.
GAWLER, Stanley William, of Wellington, New Zealand.
GRAFF, Samuel, B.Sc.(Eng.), London, of London.
HARTLAND, Robert Arthur, of London.
JEFFERIES, Robert Lionel, of Cape Town, South Africa.
LYCETT, Trevor, of Stafford.
McILROY, William Robert, of Dunedin, New Zealand.
MALLOCH, Trevor Stuart, of Wellington, New Zealand.
MERCER, Horace John Gerald, of Wolverhampton, Staffs.
MICHAU, Peter St. Cyr, of Bulawayo, Southern Rhodesia.
MOTTERSHEAD, Geoffrey, B.Sc.(Tech.), Manchester, of Wilmslow, Manchester.
NIGHTINGALE, Alan Francis, of Lower Hutt, New Zealand.
PUGH, Arthur Dudley, of Rhondda, Glam.
RYELL, John, of Stafford.
SALTER, Terence Herbert, of Beckenham, Kent.
THOMAS, David Lewis, of Johannesburg, South Africa.
TURNER, Robert William, of Coulsdon, Surrey.
VAITHEESPARA, Tharmalingam, of Kankesanturai, Ceylon.
WALKER, Brian Colin, of Johannesburg, South Africa.
WARDLE, Terence Michael, of Stourport-on-Severn, Worcs.

Graduates to Associate-Members

BANERJEE, Robindra Nath, B.Sc. Lucknow, B.E. Calcutta, of Bombay, India.
BARRASS, Desmond, of Manchester.
BINNEY, Hubert Montgomery, of Wellington, New Zealand.
BINNION, Daniel Fletcher, B.Sc.(Civil), Manchester, of Salford, Lancs.
BROOM, John David, of Christchurch, New Zealand.
BROWN, Kenneth Vincent, of Ewell, Surrey.
COLLINS, Brian Thomas, A.M.I.Mun.E., of London.
DAS GUPTA, Amal., B.Sc.(Eng.), Patna, of Ranchi, India.
DEAKIN, Brian, of Northwich, Cheshire.
FAGG, Norman Talbot, of Wellington, New Zealand.
FELIX, Nicolaas, B.Sc.(Eng.), Rand, of Johannesburg, South Africa.
FORSBREY, Leonard William, of London.
GIMI, Sohrab Barjorji, B.Sc., B.E.(Civil), Bombay, of Bombay, India.
GOLDSTEIN, Adolf, B.Sc.(Eng.)(Hons.) London, D.I.C., of London.
HEMINGWAY, Geoffrey Truscott, B.Sc.(Eng.), Cape Town, of Bulawayo, Southern Rhodesia.
HOLLENBACH, Carel Arnold, B.Sc.(Eng.), Rand, of Johannesburg, South Africa.
MUIR, Peter, of Salisbury, S. Rhodesia.
QUINION, David William, B.Sc.(Eng.)(Hons.), London, of Rickmansworth, Herts.
RAIJI, Prithviraj Diliprai, B.E.(Civil), Bombay, of Bombay, India.
REY, Jean Joseph Raymond, B.Sc.(Eng.), Rand, of Salisbury, S. Rhodesia.
SMITH, Alexander, of Salford, Lancs.
STOCKS, Hugh Gerald, of Pretoria, South Africa.
STROUDE, Clifford Herbert, B.Sc.(Civil), Cape Town, A.M.I.C.E., of East London, South Africa.
TAYLOR, Alan, of Accra, Gold Coast.
WARK, Donald Hulme, of Manchester.
WEST, Joseph Frederick, of Toronto, Canada.
WOLSTENCROFT, Derek, B.Sc.(Eng.), London, of St. Albans, Herts.

Associate-Members to Members

BILIMORIA, Rustum Kawasji, B.E., of Peradeniya, Ceylon.
EDGE, James Harold, of Johannesburg, South Africa.
MITCHELL, Denis Noel, A.M.I.C.E., of London.
RAVELLI, Filippo Luigi, of Johannesburg, South Africa.

Members to Retired Members

FRANCIS, Leonard Philip, of Edenbridge, Kent.
TROUP, Francis Gordon, F.R.I.B.A., of Henley-on-Thames.

OBITUARY

The Council regret to announce the deaths of William Adam CRAIG, Nelson FYFE, Alan Eastwood FLETCHER, Ralph Restall GARDINER, Leonard Jackson SPEIGHT, Jacob Stephenson STOUT, Louis Frederick SUMMERFIELD (Members) ; Bertram Fothergill CROSFIELD (Associate) ; Frederick Fisher CHRISTIAN, Joseph Clarkson FORREST, George Skeffington Hilne GRIMMER, George Eugene ILLASHEVITCH, Sidney Hales LEWIS, James WARD (Associate-Members).

EXAMINATIONS—JULY, 1951

OVERSEAS CENTRES

The examinations were held overseas in July, 1951, at the following centres :—Auckland, Baghdad, Bombay, Brisbane, Bulawayo, Cairo, Calcutta, Cape Town, Christchurch (N.Z.), Colombo, Delhi (Aligarh), Dunedin, Durban, East London (S.A.), Hong Kong, Jerusalem, Johannesburg, Karachi, Kuala Lumpur, Lagos, Lahore, Lucknow, Madras, Malta, Montreal, Nairobi, Port Elizabeth, Salisbury (S. Rhodesia), Shillong, Singapore, Sudan, Sydney, Tel-Aviv, Toronto, Wellington (N.Z.).

Thirty-nine candidates took the Graduateship Examination and sixty-seven the Associate-Membership Examination, making a total of one hundred and six. Of these, twenty passed the Graduateship Examination and twenty-four passed the Associate-Membership Examination.

The names of the successful candidates are :—

GRADUATESHIP EXAMINATION

ATTWOOD, William Enoch Edward; BANERJEE, Hari Har; DOYLE, William Sherwood; FAYYAZ, Ali Shah; GAWLER, Stanley William; HAQUE, Chowdhary Abdul; JEFFERIES, Robert Lionel; KLOPPER, Stephanus; McILROY, William Robert; MALLOCH, Trevor Stuart; MICHAU, Peter St. Cyr; NIGHTINGALE, Alan Francis; NOHR, Max; RAO, Akunuri Balaji; SHVARTZ, David Itzhak; THOMAS, Thomas Arthur Charles; TSAO HSIEN HWA; VAITHEESPARA, Tharummalingam; WALKER, Brian Colin; WHALE, John Frederick.

ASSOCIATE-MEMBERSHIP EXAMINATION

AKERKAR, Anand Ganesh; BINNEY, Hubert Montgomery; BROOM, John David; CUTLER, Kevin Mulqueen; DAHM, Hans; FAGG, Norman Talbot; FALISZEWSKI, Stefan Wilhelm; FELIX, Nicolaas; GIMI, Sohrab Barjorji; HEMINGWAY, Geoffrey Truscott; HENRY, Robert Nelson; MADAN, Madhukar Yeshwant; MINHAS, Mohammed Hussain; MUIR, Peter; NEWMAN, Wolfgang Max; RAFE, Abdur; RAIJI, Prithviraj Diliprai; REY, Jean Joseph Raymond; SAVONA, Joseph Sigismund; STOCKS, Hugh Gerald; STROUDE, Clifford Herbert; TARAPOREWALLA, Jalejer Jehangir; WEST, Joseph Frederick; WILLIAMSON, James Wilson.

PRIZE LIST

JULY, 1951—EXAMINATIONS

The Council have awarded the following prizes in connection with the examinations held in July, 1951 :—

ANDREWS PRIZE. (For the candidate who obtains the highest aggregate of marks in the Associate-Membership Examination, passing in all subjects.)
Samir Kumar MALLICK, of London.

HUSBAND PRIZE. (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper "Structural Engineering Design and Drawing".)
Leon John MARSHALL, of Ilford.

WALLACE PREMIUM (SENIOR). (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper "Theory of Structures (Advanced)".)
Robert Nelson HENRY, of Auckland.

WALLACE PREMIUM (JUNIOR). (For the most successful candidate in the Graduateship Examination, passing in all subjects.)
Gerard KILEY, of Swansea.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, January 24th, 1952

Ordinary General Meeting, 5.55 p.m. This meeting, which is for the election of members, and is open only to corporate members of the Institution, will be followed by an Ordinary Meeting at 6 p.m., when Mr. Arthur Bolton, B.Sc. (Graduate), will give a paper on "A New Approach to the Elastic Analysis of Two Dimensional Rigid Frames."

Thursday, February 14th, 1952

Ordinary Meeting at 6 p.m., when Mr. D. I. Lawson, M.Sc., M.I.E.E., Mr. C. T. Webster, F.R.I.C., and Mr. L. A. Ashton, B.Sc., will give a paper on "The Fire Endurance of Timber Beams and Floors."

Thursday, February 28th, 1952

Ordinary General Meeting for the election of members 5.55 p.m., followed by an Ordinary Meeting at 6 p.m. when Dr. W. B. Dobie, M.Sc., F.R.S.A., A.M.I.C.E., A.M.I.Mech.E., will give a paper on "The Torsional Strength of Structural Members."

Thursday, March 13th, 1952

Ordinary Meeting at 6 p.m., when Mr. P. G. Bowie, A.M.I.C.E. (Member), will give a paper on "Faults in Concrete Structures."

Wednesday, March 19th, 1952

Joint Meeting with the Reinforced Concrete Association, at 6 p.m., when Mr. F. S. Snow, M.I.C.E., M.I.Mech.E. (Past-President), will give a paper on "Recent Industrial Developments at Port Sunlight and Bromborough."

Thursday, March 27th, 1952

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. F. R. Bullen, B.Sc., M.I.C.E. (Member of Council) will give a paper entitled "Unusual Design for a large Constructional Shop."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS, 1952

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on January 8th and 9th, 1952 (Graduateship), and on January 10th and 11th (Associate-Membership).

REPRESENTATION

The Council have made the following nominations of members to represent the Institution :—

UNION OF LANCASHIRE AND CHESHIRE INSTITUTES
Building Advisory Committee :—
Mr. W. E. Kelsey (Associate).
Engineering Advisory Committee :—
Mr. F. Simpson (Associate-Member).

EARTH RETAINING STRUCTURES CODE

Civil Engineering Code of Practice No. 2—Earth Retaining Structures, issued by the Institution of Structural Engineers on behalf of the Civil Engineering Joint Codes Committee will be obtainable from the Institution from the second week in January, price 15s. post free to members and others.

JOURNAL CASES AND BINDING, 1951

A binding case can be supplied for the twelve issues of the Journal, January-December, 1951 (Volume 29), price 11s., post free.

The price for binding volumes is 26s. per volume, inclusive. This price is for the half-leather binding which has been in use for some years.

It is requested that all parcels and Journals forwarded for binding should bear the name, address and rank of the member concerned. All volumes for binding must be despatched to the Institution by March 31st, 1952.

An Index will be included in all volumes bound. This Index will not be generally distributed, but members and others wishing to have a copy should apply to the Secretary.

THE MACLACHLAN LECTURE

GENERAL CONDITIONS

Through the generosity of Mr. John MacLachlan (Retired Member), the Council was able in 1948 to institute an Annual Lecture to be competed for by Associate-Members. The conditions of the presentation are as follows :—

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture and to be held annually.
2. The subject of the Lecture may be on any aspect of Structural Engineering so long as in every second year the subject shall be confined to steel structures.
3. Entrance into the competition for the Lecture shall be confined to Associate-Members of the Institution, who are under the age of 32 years.
4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.
5. No paper submitted shall have been published or read elsewhere.
6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s. 0d.
7. Should a competitor's paper be considered worthy of ranking second in merit, he will receive a consolation award of £5.
8. In the event of there being no winner of the competition in any one or more years, whether because no lecture is submitted or because no lecture submitted is considered to be of sufficient merit to warrant an award, or for any other reason, the Institution shall transfer these sums to the Research Fund of the Institution.

PARTICULARS OF THE COMPETITION FOR 1952

1. The MacLachlan Lecture will be given at a meeting of ne Institution to be arranged towards the end of 1952.

2. The subject of the Lecture shall be on any aspect f structural engineering.

3. The work should be submitted as the script of a ecture which the author, if successful in the competition, ill deliver before an audience in the course of about ne hour. The development of mathematical formulæ nd detailed calculations should be avoided as far as ossible in the text ; if they are essential they should be mbodied in appendices. Photographs, drawings, graphs, tc., which would appear as illustrations to the lecture in ublished form, should accompany the script. If dditional illustrations would be shown as slides, a list f these should be included.

Lectures should be prepared in accordance with the equirements of the Literature Committee for publication n THE STRUCTURAL ENGINEER. Candidates may obtain copy of these requirements on application to the ecretary.

4. Six copies of each Lecture should be submitted and hould be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the nstitution is Monday, March 31st, 1952.

RESEARCH AWARDS

The Council have instituted a Research Prize Fund, rom which awards may be made annually to the author r joint authors of papers describing original research vhich they have carried out. Research awards may e made for papers read at Headquarters or in the Branches and published in the Journal, or for papers ublished in the Journal only without being read at an pen meeting.

The assessment for such awards will be made annually, ut awards will be made only to the contributors of such apers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original nd may include any of the following :—

(a) investigations of an experimental or analytical haracter ;

(b) studies of historical or statistical records ;

(c) improvements in principles or methods of contruction ;

(d) research into methods of structural engineering nd building, the nature and use of plant and the rganisation of engineering work ;

(e) any related or combined studies which are deemed y the Literature Committee to be of a research haracter.

In cases where the research work described in the aper was not the work of one individual, the names of ll the collaborators should be given in the paper.

Awards may take any or all of the following forms :— A research medal ; a diploma ; a money prize.

Application for consideration for a research award nust be made to the Secretary of the Institution, and in reparing papers for reproduction in the Journal, authors nust comply with the conditions laid down for all such ontributions. Particulars of these conditions may be btained from the Secretary.

In judging research papers, the following factors will e considered :—

(a) the nature of the subject and its conclusions ;

(b) the value of the paper in advancing the science nd art of structural engineering ;

(c) the standard of preparation and orderly arrangenent of the subject matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1951, and September, 1952, is October 31st, 1952.

EXAMINATIONS

PREPARATION FOR THE EXAMINATIONS OF THE INSTITUTION BY ATTENDANCE AT TECHNICAL COLLEGES

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

PREPARATION FOR THE GRADUATESHIP EXAMINATION

Technical colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

PREPARATION FOR THE ASSOCIATE-MEMBERSHIP EXAMINATION

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering ; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in the first category provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

The Colleges which have informed the Institution that courses in Structural Engineering are available are :—

- Belfast College of Technology.
- Birmingham Central Technical College.
- Bolton Municipal Technical College.
- Bradford Technical College.
- Derby Technical College.
- Dudley and Staffordshire Technical College.
- Glasgow Royal Technical College.
- City of Liverpool College of Building.
- L.C.C. Brixton School of Building.
- L.C.C. Hammersmith School of Building and Arts and Crafts.
- Manchester College of Technology.
- Middlesbrough Constantine Technical College.
- Salford Royal Technical College.
- South-West Essex Technical College, Walthamstow, E.17.
- Stockport College for Further Education.

Colleges in the second category provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in

Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete. The colleges which have informed the Institution that such courses are available are :—

Brighton Technical College.
Cardiff Technical College.
Huddersfield Technical College.
Leeds College of Technology.
London Battersea Polytechnic.
London Northampton Polytechnic.
L.C.C. Westminster Technical College.
Plymouth and Devonport Technical College.
Preston Harris Institute.
Wigan Mining and Technical College.
Woolwich Polytechnic.

Students attending colleges in the first category are advised to take the organised courses in Structural Engineering. Students of Graduate Membership standard will usually be allowed to select subjects from courses provided by colleges in the second category.

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, on Tuesday, January 29th, 1952, at 6 p.m., when an address will be given by the President of the Institution, Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E.

Hon. Secretary : D. B. Rogers, 4, Portland Rise, Finsbury Park, N.4.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Tuesday, January 22nd, 1952

Annual Dinner and Dance, at Longford Hall, Stretford, at 6 p.m.

Wednesday, January 30th, 1952

Three short lectures by Mr. W. H. Rosier, Mr. A. S. Sinclair and Mr. C. Thirsk (Associate-Members), followed by discussion, at the College of Technology, Manchester, at 6.30 p.m. (preceded by tea at 5.45 p.m.)

Thursday, February 14th, 1952

Joint Meeting with The Institution of Civil Engineers North-Western Association, at the Engineers' Club, 17, Albert Square, Manchester, at 6.30 p.m. Mr. Gilbert Roberts, B.Sc., M.I.C.E., on "The Dome of Discovery—Festival of Britain Site."

Monday, February 25th, 1952

Mr. Arthur Bolton, B.Sc. (Graduate), on "A New Approach to the Elastic Analysis of Two Dimensional Rigid Frames," at the College of Technology, Manchester, 6.30 p.m.

Wednesday, March 12th, 1952

Joint Meeting with the Liverpool Engineering Society, at The Temple, 24, Dale Street, Liverpool, at 6 p.m. Mr. J. Cunningham, B.Sc., A.M.I.C.E., on "The Britannia Tubular Bridge over the Menai Straits."

Wednesday, March 19th, 1952

Dr. G. G. Meyerhof, M.Sc., A.M.I.C.E., F.G.S. (Associate-Member), on "Some Aspects of Soil Mechanics with reference to Foundations," at the College of Technology, Manchester, 6.30 p.m.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 28, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Friday, January 25th, 1952

"Notes on Soil Mechanics," by Dr. J. Kolbuszewski, F.G.S., at James Watt Memorial Institute, Birmingham, 6 p.m.

Monday, February 25th, 1952

"Some Aspects of Soil Mechanics with reference to Foundations," by Dr. G. G. Meyerhof, M.Sc., A.M.I.C.E., F.G.S. (Associate-Member), at Derby, 7 p.m.

Friday, March 28th, 1952

"The Research Station of the Cement and Concrete Association," by Mr. P. B. Morrice, B.Sc.(Eng.), at Stafford.

Hon. Secretary : E. R. Deeley, A.M.I.Struct.E., Arranmoor, Adshead Road, Dudley, Worcs.

GRADUATES' AND STUDENTS' SECTION

The Annual General Meeting of the Section will be held on Wednesday, January 30th, 1952, at 7 p.m., at the James Watt Memorial Institute, Birmingham, and will be followed by a film and a talk on "Welding as Applied to Structural Engineering," by Mr. F. Brooksbank, B.A.(Graduate).

Hon. Secretary : H. M. Evans, B.Sc., 42, Church Hill Road, Handsworth, Birmingham, 20.

NORTHERN COUNTIES' BRANCH

The following meetings have been arranged :—

Wednesday, January 9th, 1952

Joint Meeting with the Institution of Civil Engineers, at the Cleveland Scientific and Technical Institution, Middlesbrough. Mr. W. R. Garrett, A.M.I.C.E. (Associate-Member), on "Reconstruction of Houdon-on-Tyne Gas Works."

Wednesday, January 16th, 1952

The above meeting will be repeated at the Neville Hall, Newcastle.

Tuesday, February 5th, 1952

Mr. L. Scott White, O.B.E., M.I.C.E. (Past-President), on "The Moving of King Henry VIII's Wine Cellar, Whitehall Gardens," at Middlesbrough.

Wednesday, February 6th, 1952

The above meeting will be repeated at Newcastle.

Tuesday, March 4th, 1952

Mr. G. S. Gowland (Associate-Member), on "Impressions of U.S.A. Welding Methods," at Middlesbrough.

Wednesday, March 5th, 1952

The above meeting will be repeated at Newcastle.

All meetings will commence at 6.30 p.m., preceded by tea at 6 p.m.

Hon. Secretary : Ian MacGregor, M.I.Struct.E., 9, Ellison Place, Newcastle-upon-Tyne, 1.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Wednesday, January 23rd, 1952

Annual Dinner and Social Function.

Tuesday, February 5th, 1952

Mr. R. Montgomery (Associate-Member), on "Some Economies in the Fabrication of Steel Sections," at the College of Technology, Belfast, 7.30 p.m.

Tuesday, March 4th, 1952

Mr. H. M. Nelson, B.Sc., on "Plastic Design applied o Structural Engineering," at the College of Technology, 3elfast, 7.30 p.m.

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., 'Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Wednesday, January 16th, 1952

Mr. J. Guthrie Brown, M.I.C.E. (Member of Council), on "The Tummel-Garry Hydro-Electric Scheme," at the Ca'doro Restaurant, 6.0 p.m.

Wednesday, February 13th, 1952

Mr. J. Dixon and Mr. D. M. Campbell, B.Sc.(Graduate) on "Site Exploration and Rock Drilling Methods," at the Ca'doro Restaurant, 6.0 p.m.

Tuesday, March 11th, 1952

Mr. W. A. Fairhurst (Member) on "The Design of Engineering Structures including Concrete Bridges," at the Ca'doro Restaurant, 6.0 p.m.

Hon. Secretary : D. G. Drummond, B.Sc., A.M.I.C.E., M.I.Struct.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

The next meeting of the Branch will be held at Newton Abbot on Friday, January 11th, 1952, when Mr. Wallace A. Evans (Member) will give a paper on "The Reconstruction of Abertillery Bridge."

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10/12, Market Street, Torquay.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, February 13th, 1952

Mr. Wallace A. Evans (Member), on "The Completed Abertillery Bridge," at the Mackworth Hotel, Swansea, 6.30 p.m.

Friday, February 15th, 1952

A meeting will be held at Colwyn Bay, details of which will be announced later.

Tuesday, February 19th, 1952

Mr. Wallace A. Evans (Member), on "The Completed Abertillery Bridge," at the South Wales Institute of Engineers, Cardiff, 6.30 p.m.

Tuesday, March 4th, 1952

Combined meeting with the South Wales Association of The Institution of Civil Engineers, at Cardiff. Films will be shown.

Wednesday, March 5th, 1952

A meeting will be held at Swansea, when the films referred to above will be repeated.

Friday, March 21st, 1952

A meeting will be held at Colwyn Bay, details of which will be announced later.

Hon. Secretary : E. R. Steward, A.M.I.Struct.E., Edrom, Ashleigh Road, Blackpill, Swansea.

WESTERN COUNTIES BRANCH

The second Branch Meeting of the Session was held in the Reception Room, Bristol University, on Thursday, November 1st, 1951, at 5.30 p.m. The meeting was held jointly with the South-Western Association of The Institution of Civil Engineers, whose Chairman, Mr. J. B. Bennett, presided over a large gathering.

Mr. E. Bateson, M.I.C.E., M.I.Struct.E., gave a paper on "Some interesting problems in Bridge Reconstruction," which comprised an account of various strengthening and rebuilding schemes for some large bridges in India. Following replies by the lecturer to a number of questions, a vote of thanks was proposed by Mr. E. N. Underwood B.Sc., A.M.I.C.E. (Member) (Branch Vice-Chairman), and seconded by Mr. N. A. Matheson.

The following meetings have been arranged :—

Friday, January 4th, 1952

Mr. O. H. Willey, on "The Fabrication of Steel Structures," at Bristol University, 6 p.m.

Friday, February 1st, 1952

Combined meeting with the Institution of Civil Engineers. Dr. A. R. Collins, M.B.E., A.M.I.C.E. (Associate-Member), on "Some Effects of Recent Developments on the Design and Construction of Concrete Structures."

Wednesday, February 20th, 1952

Annual Dinner at the Royal Hotel, Bristol.

Hon. Secretary : C. E. Saunders, M.I.Struct.E., Dunkery, Edward Road, Walton St. Mary, Clevedon, Somerset.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, January 16th, 1952

Mr. A. V. Hooker, A.M.I.C.E. (Associate-Member), on "Structural Engineering at Abbey Works," at the Great Northern Hotel, Leeds, 6.30 p.m.

Friday, February 8th, 1952

Annual Dinner and Dance, Parkway Hotel, Otley Road, Leeds, 7 p.m.

Wednesday, February 20th, 1952

Combined meeting with the Yorkshire Association of The Institution of Civil Engineers. Mr. James N. Garden, A.M.I.C.E., on "Prestressed Concrete Bridge, Skelton Grange Power Station, Leeds," at Leeds University, 7 p.m.

Wednesday, March 19th, 1952

Mr. Hugh B. Sutherland, S.M.(Harvard), A.M.I.C.E. (Associate-Member), on "Problems in Foundation Engineering," at the Great Northern Hotel, Leeds, 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days, Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone : 34-1111. Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

ADDITIONS TO THE LIBRARY

The following volumes have been added to the Library :—

BERRY, John. *Reinforced Concrete Design.* London, 1945. Presented by Mr. F. G. Etches.

BROOKS, W. H. *Strength and Elasticity of Materials and Theory of Structures,* Vol. I. London, 1950. Presented by Mr. A. E. Peatfield.

CAMM, F. J. (Editor). *Newnes' Engineer's Reference Book.* London, 1949. Presented by Mr. L. Freeborn.

CLARK, F. W. *Tube Works Gauges and Gauging Practice.* Glasgow, Birmingham and London, 1950.

ERIKSEN, B. *Influence Lines for Thrust and Bending Moments in the Fixed Arch.* London, 1947.

ESCRITT, L. B. *Sewage Treatment : Design and Specification.* London, 1950. Presented by Mr. L. J. Griffiths.

GRINTER, L. E. *Design of Modern Steel Structures.* New York and London, 1948. Presented by Mr. W. S. Watts.

HANCOCK, G. J. *Asphalte in Modern Building Construction.* London, 1950. Presented by Mr. S. C. Gibbins.

HAYDEN, A. G., and BARRON, M. *The Rigid-Frame Bridge.* 3rd Edition. New York, 1950. Presented by Mr. J. J. Leeming.

HETENYI, M. (Editor). *Handbook of Experimental Stress Analysis.* New York and London, 1950. Presented by Professor W. Fisher Cassie.

HILTON, B. R. *Welding Design and Processes.* London, 1950. Presented by Mr. P. C. G. Hausser.

HOYLE, E. *Sketching for Craftsmen.* London, 1950. Presented by Mr. S. J. Crispin.

JESSOP, H. T. and HARRIS, F. C. *Photoelasticity : Principles and Methods.* London, 1949. Presented by Mr. P. L. Capper.

KNIGHT, B. H. *Soil Mechanics for Civil Engineers.* London, 1948.

KRYNINE, D. P. *Soil Mechanics : Its Principles and Structural Applications.* 2nd Edition. New York and London, 1947. Presented by Mr. M. M. Khann.

LEE, G. H. *Introduction to Experimental Stress Analysis.* London, 1950. Presented by Mr. F. B. Bull.

LEWITT, E. H. *Definitions and Formulæ for Students : Applied Mechanics.* 2nd Edition. London, 1950.

LUNDGREN, H. *Cylindrical Shells.* Vol. I—*Cylindrical Roofs.* Copenhagen, 1949.

MICHAELS, L. *Contemporary Structure in Architecture* New York, 1950. Presented by Mr. A. H. Ley.

MINIKIN, R. R. *Winds, Waves and Maritime Structures* London, 1950. Presented by Mr. M. Nachshen.

NASH, K. Ll. *The Elements of Soil Mechanics in Theory and Practice.* London, 1951. Presented by Mr. C. B Brown.

O'SULLIVAN, T. P. *The Economic Design of Rectangula Reinforced Concrete Sections.* London, 1950. Presented by Mr. P. G. Bowie.

Quebec Bridge Inquiry. Report of the Royal Commission and Plans, Ottawa, 1908. Presented by Mr. H. G Cownley.

SALIGER, R. *Die neue Theorie des Stahlbetons auf Grund der Bildsamkeit vor dem Bruch.* Vienna, 1950 Presented by Mr. B. A. E. Hiley.

SALIGER, R. *Der Stahlbetonbau : Werkstoff, Berechnung und Gestaltung.* Vienna, 1949. Presented by Dr. P W. Abeles.

SCOTT, W. L., GLANVILLE, W. H., and THOMAS, F. G *Explanatory Handbook on the B.S. Code of Practice fo Reinforced Concrete.* London, 1950.

SMITHSONIAN INSTITUTION. *Annual Report,* 1949. Washington, 1950.

STEED, R. W. *An Introduction to Distribution Method of Structural Analysis.* London, 1950. Presented by Mr. C. H. Hockley.

WOOD, R. D. *Junior Principles of Quantity Surveying* London, 1950. Presented by Mr. H. H. B. Stewart.

YOUNG, J. McHardy. *Structural Theory and Design* Vol. I. London, 1950. Presented by Mr. D. T Williams.

The following publications have been presented by Mr. P. S. Pandit :—

BAKER, A. L. L. *Reinforced Concrete.* London, 1949.

CROSS, Hardy, and MORGAN, N. D. *Continuous Frame in Reinforced Concrete.* 11th Printing. New York 1949.

Institution of Civil Engineers. *Report of Proceedings o Conference on Pre-Stressed Concrete.* London, 1949.

MINIKIN, R. R. *Structural Foundations.* London, 1948

REYNOLDS, T. J., and KENT, L. E. *Introduction t Structural Mechanics for Building and Architectura Students.*

Book Review

Contemporary Structure in Architecture, by Leonard Michaels. (New York : Reinhold Publishing Corporation.)

This book is a refreshing contribution to architecture from an engineering aspect, and a refreshing contribution to engineering from an architectural aspect.

It records in a most interesting manner the development of modern structural principles, and shows the way in which this development has, or should have, affected modern architectural design.

One is rather surprised that the opportunities which exist for using stressed skin construction and box frames have not been more widely developed in this country, and this book forms an admirable reference to demonstrate the great possibilities which exist for the employment of these design theories.

The 247 illustrations are well chosen and show the advances which have been made in "Contemporary Structure in Architecture," and the descriptive matter carries one's thoughts forward through posts and lintels, portal frames, cantilevers and space frames, to th lighter forms of construction rendered possible by monolithic stressed skins. It is, however, pleasan after having studied many pages and over 200 illustrations of advance methods of construction, to come upon pictures of such wonderful examples of buildings of th past as the Colosseum in Rome, and the Guildhall a Thaxted.

From the use of scientific principles, there is emergin to-day a new moulded or plastic architectural styl which reveals a potential greatness to rival its tw classical predecessors, the "Trabiated" and th "Arched" methods of construction.

The contribution which this book makes to moder ideas in building is its demonstration of how every par of a structure can be made to contribute its maximur efficiency, and illustrations of the single span bridge demonstrate the grace and beauty which this ca produce.

A. H. L.

The Fire Endurance of Timber Beams and Floors*†

By D. I. Lawson, M.Sc., M.I.E.E., F.Inst.P., C. T. Webster, F.R.I.C. and L. A. Ashton, B.Sc.

Synopsis

The fire endurance‡ of a variety of timber beams under a range of loading conditions has been measured when the beams were subjected to fire resistance tests, as defined in B.S. 476-1932. From observations of the rate of charring of the beams it was found that their fire endurance could be found from the following expression :

$$r^{\frac{1}{4}} = \tfrac{1}{2}\,(1 - T\sqrt{s})\,(1 - T/2\sqrt{s})^2$$

where

r is the ratio $\dfrac{\text{applied load}}{\text{breaking load}}$

s is a shape factor $= \dfrac{\text{depth of beam}}{\text{breadth of beam}}$

$T = t/20\sqrt{a}$

a is the area of cross section of the beam

t is its fire endurance.

Curves have been plotted giving the fire endurance of most common sizes of beam in terms of the load conditions, and it has been shown that for similar shaped beams under equivalent load conditions the fire endurance is proportional to the square root of the cross-sectional area. For a given cross-sectional area and corresponding fibre stress, a beam having a square section will give the longest fire endurance.

The analysis has been extended to predict the fire endurance of floors supported on timber beams by allowing for the time taken for the fire to penetrate the ceiling supporting the beams. When this time is added to the time taken by the beam to fail, the result is in good agreement with experimental measurements made on the fire endurance of such floors.

I. Introduction

The need has long been felt for a method of assessing the fire endurance of timber beams under various intensities of stress. This information is required for the design of floors and roofs, and recently the need for timber economy has led to proposals that the shape of floor joist be altered to permit the use of timber sections of smaller area. The permissible reduction in size will be governed by the fire endurance of such beams when loaded. In order to rectify the deficiency a start was made between the years 1945-6 at the Building Research Station with a series of fire resistance tests as defined in B.S. 476-1932[1] on timber floors of various constructions. The results of these experiments were of considerable value as design data ; they do not however cover the variety of conditions encountered in practice, and the present paper shows how they may be used to derive a formula by means of which the fire endurance of any timber joist system can be estimated. The work is semi-empirical in that a theoretical expression for the fire endurance is derived making certain assumptions, and the results of the experiments are compared with the predicted performance. The original expression is then modified to fit the observed results more closely.

II. Analysis of the Fire Endurance of Beams

It is not possible from existing knowledge to derive an exact expression relating the performance of a beam under fire conditions to its dimensions and to the applied load. A rigorous analysis would require a knowledge of the temperature conditions throughout the beam during the test, and these would have to be related to the strength of the timber for the various temperatures encountered. Even if such an expression were derived it would almost certainly be too complicated to use. An alternative approach would be to make certain simplifying assumptions and to compare the results obtained in this way with practical experience. The original expression may then be modified to fit the experimental results. This procedure has been adopted during the present work.

It has been observed in the experiments to be outlined that during a fire test as described in B.S. 476 the line of demarcation between the charred and uncharred portion of the timber beam is well defined, and this advances at an average rate of about 1/40 in./min. If the assumption is made that the timber retains its original strength until charring takes place, and after this the strength falls to zero, it is possible to calculate the strength of the beam at any time during the test. If the beam has initial dimensions b and d representing respectively the breadth and the depth in inches, then after a time t minutes, these will have been reduced to $(b - t/20)$ and $(d - t/40)$ since the charring will have taken place on the underside and on both faces of the beam, the upper surface being protected by the floor. The load W_t which after t minutes would just cause failure in bending is given by

$$W_t = k\,(b - t/20)\,(d - t/40)^2 \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where k contains such factors as the span of the beam and the ultimate fibre stress§ of the beam material. Comparison of the strength of the beam after it has been subjected to fire conditions for a time t with its original strength W_o gives

$$r = W_t/W_o = (b - t/20)\,(d - t/40)^2/bd^2$$
$$= (1 - t/20b)\,(1 - t/40d)^2 \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Thus, if a beam is loaded to a fraction $r = W_t/W_o$ of its breaking load or modulus of rupture, it will fail after a time t given by expression (2).

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*Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 14th, 1952, at 6 p.m.

‡The term fire endurance for the purpose of this paper is used to denote the time taken by the structural element to collapse under fire conditions. This should be distinguished from the term fire resistance as defined in B.S. 476-1932, which imposes the further conditions that the floor should resist flame penetration and should have a limited temperature rise on its upper surface.

§A value of 11,000 lb./in.² has been assumed for the extreme fibre stress in bending at the maximum load. (G. A. Mitchell, Building Construction, Part 2. Batsford, Ltd., Thirteenth Edition, 1943, table opposite page 198).

It will be convenient for subsequent discussions of the effect of the shape of the beam on its fire endurance to introduce two new variables into expression (2) : s the cross-sectional shape ratio d/b, and a the area of cross-section db.

These give :

$$d = \sqrt{sa}, \quad b = \sqrt{(a/s)}$$

Expression (2) therefore becomes

$$r = \left(1 - \frac{t}{20}\sqrt{\frac{s}{a}}\right)\left(1 - \frac{t}{40\sqrt{sa}}\right)^2 \quad \ldots (3)$$

Since this expression is a function of $t/\sqrt{a}$ it follows that $t \propto \sqrt{a}$ for all beams of a given shape and under equivalent load conditions, i.e., the fire endurance should be linearly related to the square root of the area of cross-section. Another variable $T = t/20\sqrt{a}$ may be introduced and expression (3) becomes

$$r = (1 - T\sqrt{s})\,(1 - T/2\sqrt{s})^2 \quad \ldots \ldots (4)$$

Before elaborating this theory it will be prudent, in view of the tentative assumption on which it is based, to compare the predicted times of failure of the beam with those obtained in actual tests.

III. Experimental

A number of floor sections each comprising two Douglas fir joists stiffened with herring-bone strutting at the centre (Fig. 1), and having the dimensions shown in Table 1, were simply supported over a furnace, the temperature of which was raised in accordance with the time-temperature curve described in B.S. 476 representing the conditions obtaining in a fire. The beams

Timber beam before fire test

Timber beam after fire test

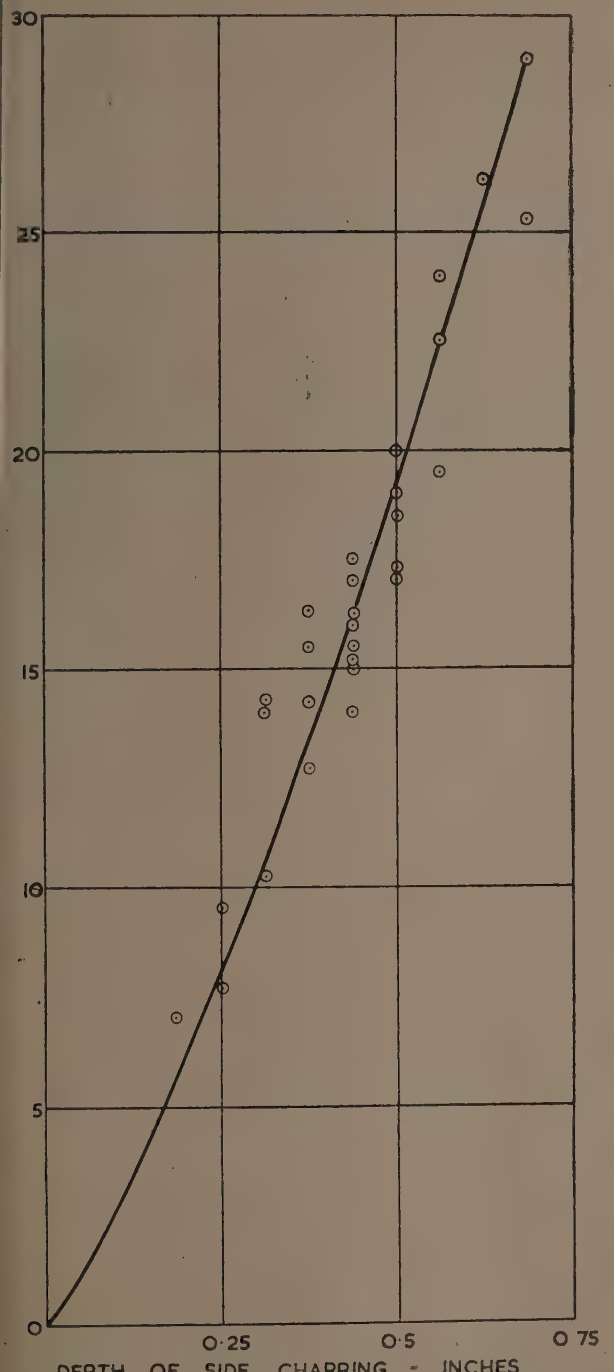
Fig. 1 [Crown copyright reserved

TABLE 1.—Results of First Series of Fire Endurance Tests on Beams of Douglas Fir

Specimen No.	Actual size of beams (in.)	Stress grading* Joist 1	Stress grading* Joist 2	Working stress (lb./in.^{2})	Breaking load factor r	Fire endurance (min.)
Nominal 9 in. × 2 in.	8⅞ × 1 13/16	—	—	835	0.0759	16¼
	8¾ × 1 29/32	—	—	200	0.0182	24
	8¾ × 1 15/16	—	1200	400	0.0364	22½
	8¾ × 2	—	1200	600	0.0546	15
	8¾ × 2	1200	—	600	0.0546	20
	8¾ × 1 15/16	1600	1200	200	0.0182	29
Nominal 7 in. × 1½ in.	6 13/16 × 1 11/32	1200	1600	856	0.0778	7¾
	6 11/16 × 1⅜	—	—	200	0.0182	17
	6¾ × 1⅜	1200	1200	400	0.0364	17¼
	6 13/16 × 1 5/16	1200	1600	600	0.0546	14¼
	6 11/16 × 1 5/16	1200	1200	600	0.0546	15½
	6 11/16 × 1 13/32	1600	—	400	0.0364	16¼
Nominal 6 in. × 2 in.	5⅞ × 1 29/32	1600	1600	859	0.0781	15½
	5¾ × 1⅞	1200	1200	200	0.0182	26¼
	5⅞ × 1 15/16	1200	1200	400	0.0364	19
	5⅞ × 1⅞	1600	—	600	0.0546	17½
	5⅞ × 2	1200	—	600	0.0546	16
	5⅞ × 1 15/16	1600	1200	400	0.0364	18½
Nominal 5 in. × 1½ in.	5 × 1 13/32	1200	1200	887	0.0806	14
	4 15/16 × 1 11/32	1200	1200	200	0.0182	15¼
	5 × 1 11/32	1600	1600	400	0.0364	12¾
	5 × 1⅜	—	1200	600	0.0546	10¼
	5 × 1 13/32	—	—	600	0.0546	9½
	5 × 1⅜	—	—	800	0.0728	7
Nominal 4 in. × 2 in.	3 11/16 × 1 31/32	1600	1200	917	0.0833	14¼
	3 13/16 × 1 31/32	1200	—	200	0.0182	25¼
	3⅞ × 1 15/16	1200	1200	400	0.0364	19½
	3 15/16 × 1 31/32	1200	1200	600	0.0546	17
	3⅞ × 1 29/32	1200	1200	600	0.0546	14
	3 13/16 × 1 15/16	1200	—	400	0.0364	21

*Where the stress grading is not given, the joist through some defect fell just below the minimum stress standard of B.S. 940 : Part 2 : 1942 (Grading schedule C).

re covered with 1 in. plain-edged boarding at ht-angles to the span, laid with $\frac{1}{8}$ in. gaps at the nts to avoid any extra contribution to the bending ength. The beams were centrally loaded, and servations were made of the time taken by the beam fail after the furnace was ignited. Fig. 1 shows a am before and after test The results of these sts are shown in Table 1. For this series of tests



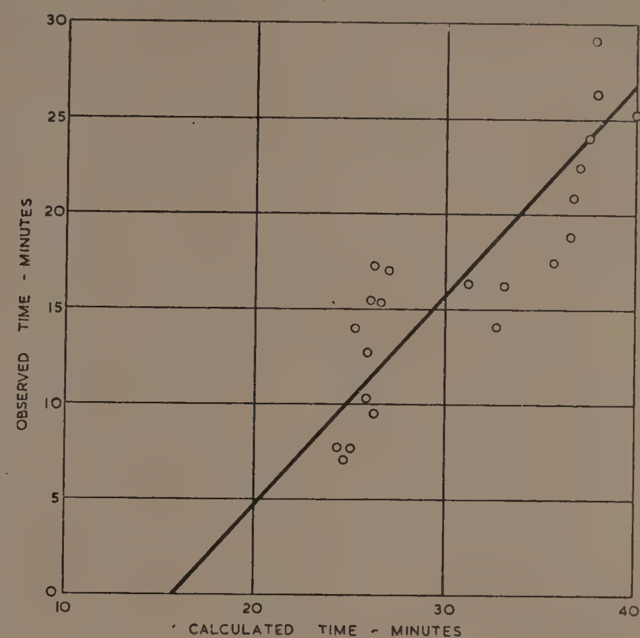
[Crown copyright reserved

Fig. 2.—Depth of charring of timber beams as a function of the duration of the fire test

e timber was not specially selected, but at a later date rther similar tests were carried out on 7 in. × 2 in. ists and, in this case, the joists were stress graded cording to B.S. 940 : Part 2 : 1942[2] to obtain members ving the range of quality from 800 lb./in.^{2} to that of ear timber. Quarter point loading was adopted to reduce the influence of chance defects and to secure greater uniformity in the results.

The experiments were repeated, using beams of 7 in. × 2 in. spruce, and again the timber was selected for these tests as above. The results are shown in Table 2.

Immediately after collapse the beams were cooled by water and observations were made on the depth of



[Crown copyright reserved

Fig. 3.—Experimental results of fire endurance of timber beam plotted as a function of values obtained from expression (3)

charring. This was always quite sharply defined. From the results obtained with the various sections it was found that the depth of charring x after a time t progressed according to the law $x = 0.051t^{0 \cdot 8}$ over the range of sizes considered; corresponding roughly to a rate of charring of 1/40 in./min. (Fig. 2.)

IV. Discussion of Results

The experimental results for the fire endurance of beams shown in Table 1 have been graphed as a function of the results computed from expression (3) in Fig. 3. The fact that the resulting graph is not a straight line with unit slope passing through the origin shows that the expression derived for the fire endurance is inaccurate. As a further approach an attempt was made to fit the results to a slightly modified expression. The new expression

$$r^n = c(1 - T\sqrt{s})\,(1 - T/2\sqrt{s})^2 \quad . \quad . \quad . \quad . \quad . \quad (5)$$

where n and c are constants to be determined by experiment gives

$$\text{nlogr} = \text{logc} + \log(1 - T\sqrt{s})\,(1 - T/2\sqrt{s})^2 \quad . \quad . \quad . \quad (6)$$

From the experimental values of Tables 1 and 2 values for $\log r$ and $\log(1 - T\sqrt{s})\,(1 - T/2\sqrt{s})^2$ may be computed, and if expression (5) is acceptable, a linear relationship should exist between these quantities. These have been graphed in Fig. 4, where it will be seen that this is indeed the case. Wherever the beams have been loaded to the same fraction of the ultimate load, i.e., have the same value of r, the mean values of $\log(1 - T\sqrt{s})\,(1 - T/2\sqrt{s})^2$ have been plotted. From Fig. 4 it is possible to obtain values for n and c ; these

TABLE 2.—Results of Second Series of Fire Endurance Tests on Timber Beams

All specimens tested to failure with quarter point loading to give extreme fibre stress of 600 lb./in.² Breaking load factor $r = 0.054$

Timber species	Size of beam (Nominal 7 in. × 2 in.) (in.)	Stress grading Joist 1	Stress grading Joist 2	Fire endurance (min.)
Douglas fir	6¾ × 1⅞	800	800	19¾
	6¾ × 1⅞	800	800	18¾
	6 23/32 × 1⅞	800	800	12¾
	6¾ × 1 27/32	—	—	19¼
	6¾ × 1 13/16	800	800	13½
	6¾ × 1⅞	800	800	13¾
	6¾ × 1⅞	Clear	Clear	20½
	6¾ × 1⅞	,,	,,	18¼
	6¾ × 1⅞	,,	,,	12
	6¾ × 1⅞	,,	,,	13½
	6¾ × 1⅞	,,	,,	19¼
Spruce	6¾ × 1⅞	—	—	16
	6¾ × 1⅞	1000	1000	15¾
	6 25/32 × 1⅞	—	—	14¼
	6¾ × 1⅞	800	800	14½
	6¾ × 1⅞	800	800	15¾
	6 25/32 × 1⅞	—	—	9¾
	6¾ × 1⅞	Clear	Clear	16¾
	6¾ × 1 27/32	1200	Clear	15
	6 25/32 × 1⅞	1200	1200	14¼
	6¾ × 1⅞	1200	1200	13¼
	6 11/16 × 1 27/32	1200	1200	14¾

both have the value of ½ and thus the expression relating the fire endurance of timber beams to their physical dimensions becomes

$$r^{\frac{1}{2}} = \tfrac{1}{2}(1 - T\sqrt{s})\,(1 - T/2\sqrt{s})^2 \quad . \quad . \quad . \quad . \quad . \quad (7)$$

There is good agreement between the experimental fire endurance of beams and those computed from expression (7) (Fig. 5). The only marked deviations occur in beams which failed soon after the test had commenced, and these may be attributed to the difficulty of controlling accurately the furnace temperature during the early stages of the test; this represents a large fraction of the time in tests of short duration.

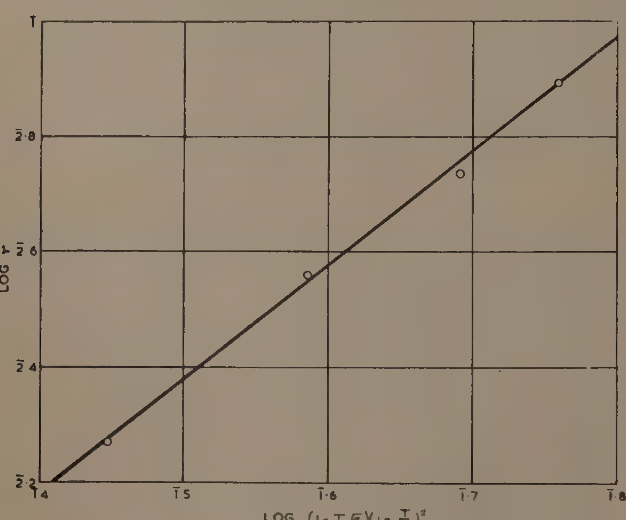


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Fig. 4.—Log r plotted as a function of log $(1—T\sqrt{s})\ (1—\frac{T_2}{2\sqrt{s}})$

where $T = \frac{t}{20\sqrt{a}}$

t = time to fail

a = area of cross section

$r = \frac{\text{load}}{\text{Breaking load}}$

$s = \frac{\text{depth of beam}}{\text{breadth of beam}}$

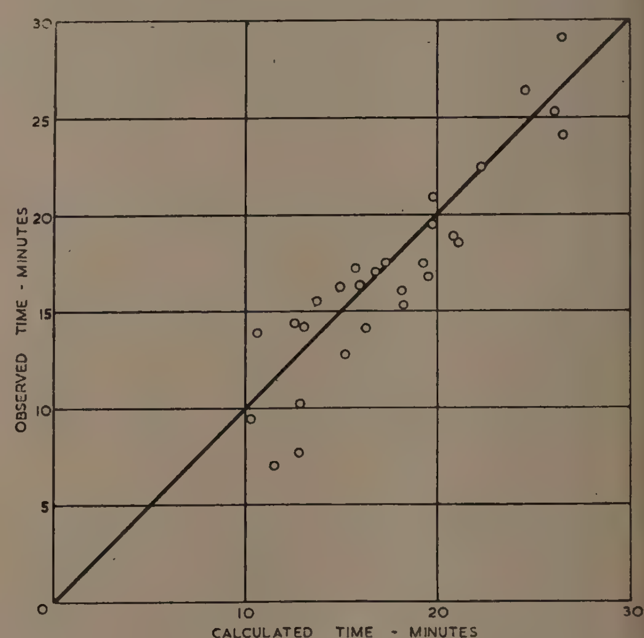

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Fig. 5.—Comparison of experimental times for fire endurance with those calculated from $r^{\frac{1}{2}} = \frac{1}{2}(1—T\sqrt{s})\ (1—\frac{T_2}{2\sqrt{s}})$

V. Optimum Dimensions of Beam for Maximum Fire Endurance

It is well known that in simple theory for a beam to have the maximum strength for a given cross-sectional area, the ratio d/b should be as large as possible. It is also clear, however, that as b is reduced the vulnerability to fire is progressively increased, so that the require-

TABLE 3.—Fire Endurance of Timber Beams of Various Sizes when Subjected to Different Loads

Size of beam (in.)	Breaking load ratio r					
	0.136	0.109	0.090	0.072	0.054	0.036
	min.	min.	min.	min.	min.	min.
9 × 3	12¾	16¼	20	23½	27¼	32¾
9 × 2	9¼	12	14¼	17	19	23
8 × 2	9	11¼	14	16½	21½	23
8 × 1½	7	9	10¾	13	14½	17¼
7 × 3	9½	15½	19	22½	26	31½
7 × 2	8¾	11	13¾	16	18¾	22½
7 × 1½	7	8¾	10¾	12¾	14½	17¼
6 × 2	8½	10¾	13¼	15¾	18¼	22
6 × 1½	6¾	8½	10½	12½	16	17¼
6 × 1	4¾	6	7¼	8¾	9¾	11¾
5 × 2	8¼	10½	12¾	15¼	17¾	21¼
5 × 1½	6½	8¼	10¼	12	13¾	16¾
5 × 1	4¾	6	7¼	8½	10	11½
4 × 2	7¾	10	12¼	14¾	17	20½
4 × 1½	6¼	8	9¾	11½	13¼	16
4 × 1	4¾	6	7	8¼	9½	11½

ments of mechanical efficiency and fire resistance are incompatible.

The shape of beam having the optimum fire resistance when loaded to a given fraction of its breaking load will be given by the maxima of T in expression 7. The analytical solution of this is both difficult and complicated, and the problem of finding the maxima of T in relation to s is more easily solved by graphical methods.* Fig. 6 shows a family of curves of T as a function of s with r as a parameter. It will be seen that T (and therefore the fire endurance) is a maximum for $s = 1$ irrespective of the loading ratio (r) of the beam. A square cross-section beam would therefore have the maximum fire endurance when loaded to a given fraction of its breaking load, and this would be about 50 per cent. greater than that given by sections in common use

For ease of computation it is better to graph r as a function of T with s as a parameter, and then to replot T as a function of s with r as a parameter.

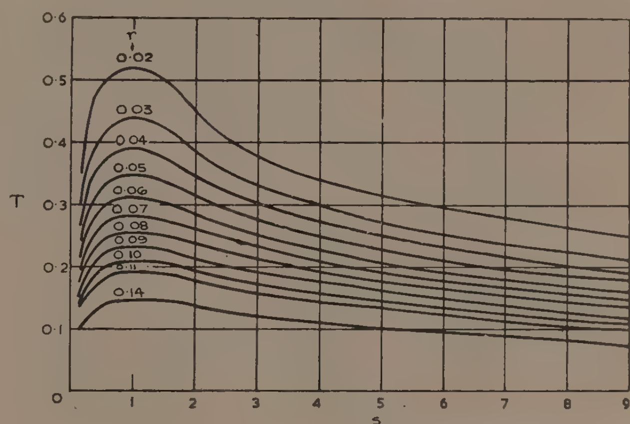


Fig. 6.—The family of curves $r^{\frac{1}{2}} = \frac{1}{2}(1 - T\sqrt{s})(1 - \frac{T_2}{2\sqrt{s}})$

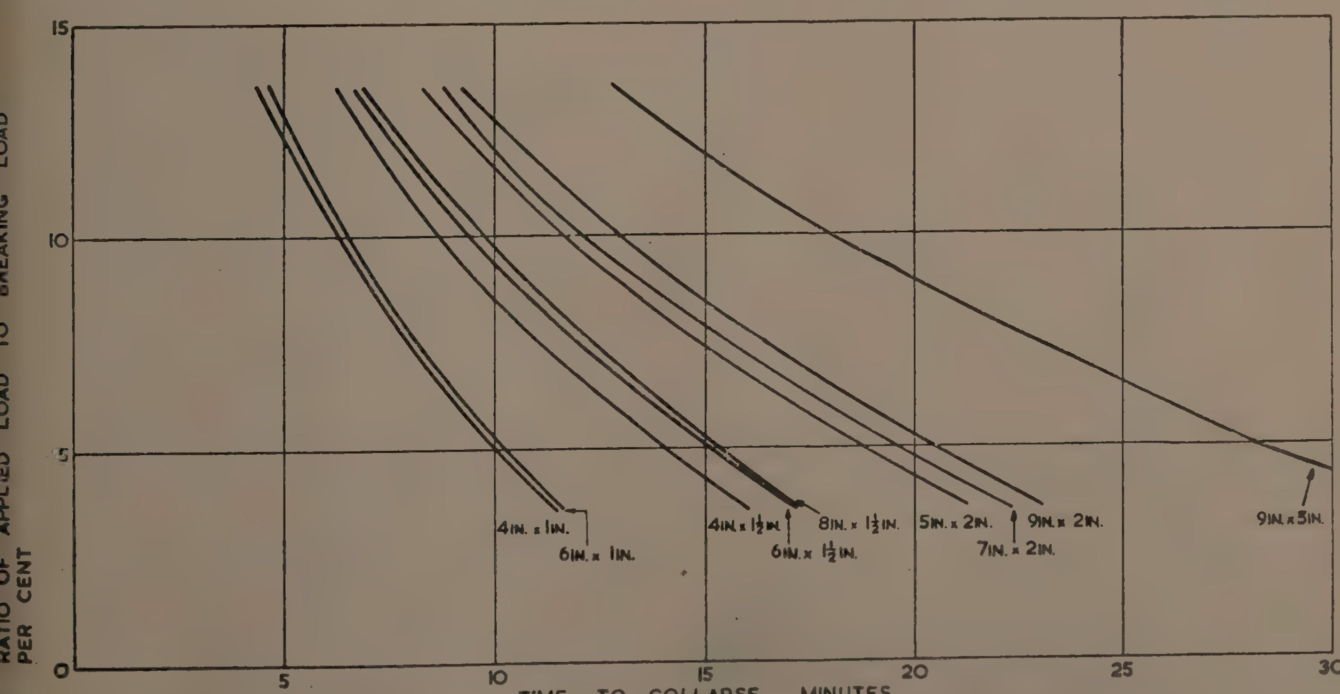


Fig. 7.—The fire endurance of various sizes of timber beams as a function of the load

($s = 3.5$ to 4.5). A square section beam is however not economical to use as the load it can support is lower than that supported by a deeper beam having the same area of cross-section. When this factor is allowed for, it may be shown that over the range of sections tested, the optimum shape is not very critical for beams having normal superimposed loads.

VI. Practical Considerations

The family of curves shown in Fig. 6 enables the fire endurance of timber beams of various sizes to be calculated rapidly, and to do this three quantities are required

(1) the area of cross-section $a = db$ in in^2.
(2) the shape ratio $s = d/b$
(3) the ratio $r = \dfrac{\text{applied load}}{\text{breaking load}} = \dfrac{\text{fibre stress under applied load}}{\text{ultimate fibre stress.}}$

Knowing r the appropriate curve is chosen from the family of curves in Fig. 6 and the value of T corresponding to the known value of s is found. The fire endurance of the beam is found by multiplying T by $20\sqrt{a}$, a being in square inches, i.e., $t = 20T\sqrt{a}$.

The fire endurance of some of the usual beam sections under different load conditions are shown in Figs. 7 and 8, and Table 3. It will be seen from these curves that the main factors governing the fire endurance of a beam are the width of the beam and the applied load.

VII. The Fire Endurance of Timber Beams with Associated Ceiling Constructions

When considering the fire endurance of floor constructions employing timber beams, additional protection is afforded by the ceiling. Table 4 shows the time taken by fire to penetrate the various ceiling constructions, and these times should be added to those for the fire endurance of the beams for the particular combination under consideration.

TABLE 4.—Time for Fire to Penetrate Various Ceiling Constructions

Description of ceiling	Time for flame penetration (min.)
Fibreboard ½ in. thick without plaster finish	8
with 3/16 in. skim coat of plaster	10
with ½ in. coat of plaster	20
Plasterboard ⅜ in. thick without plaster finish	10
with 3/16 in. skim coat of plaster	13
½ in. thick without plaster finish	18
with skim coat of plaster	25
Plaster (gypsum) ⅝ in. thick on wood lath	17
Plaster ⅞ in. on expanded metal	41

It will be seen that often it would be cheaper to obtain the requisite fire endurance by the provision of a better ceiling protection rather than by increasing the beam section.

A number of fire resistance tests were carried out on floors supported on timber beams protected by various ceiling constructions. All the specimens were supported on beams having a clear span of 12 ft. with a bearing of 5 in. at each end. The heating was in accordance with the time-temperature curve of B.S. 476.

Table 5 shows the fire endurance obtained experimentally, and this is compared with the predicted fire endurance for the timber beams (Fig. 7), added to the fire endurance of the ceiling construction as given in Table 4.

VIII. Conclusions

It has been shown that the fire endurance of timber beams may be calculated, given the dimensions and the

TABLE 5.—Comparison of Calculated Fire Endurance with Actual Test Results for Various Types of Floors

Description of specimen	Applied test load (lb./ft.²)	Time for flame penetration of ceiling from Table 4 (min.)	Endurance of joists from Figure 7 (min.)	Total estimated endurance for floor (min)	Time to collapse in test (min.)
7 in. × 2 in. joists at 15 in. ctrs.; flooring of 1 in. P.E. board; ceiling ⅜ in. plasterboard and skim coat of plaster	60	13	11½*	24½	24
ditto, but ceiling ⅝ in. gypsum plaster on wood lath	60	17	11*	28	27
ditto, but ceiling ½ in. fibre insulation board ...	60	8	12*	20	22
7 in. × 2 in. joists at 16 in. ctrs.; flooring of 1 in. T & G board; ceiling ½ in. fibre insulation board	30	8	18	26	30
9 in. × 2 in. joists at 16 in. ctrs.; flooring of 1 in. T & G board; ceiling ½ in. fibre insulation board and ½ in. plaster	60	20	17	37	43
ditto, but ceiling ½ in. plasterboard	60	18	17	35	33

NOTE.—Where tongued-and-grooved boarding is used an increase in fire endurance is to be expected owing to its stiffening effect on the joists. In the experimental constructions the joists were stiffened at the centre of the span by herring-bone strutting; no comparative tests were made of joists without strutting.

The difference between the figures marked with an asterisk* is due to the difference in the dead weights of the ceilings.

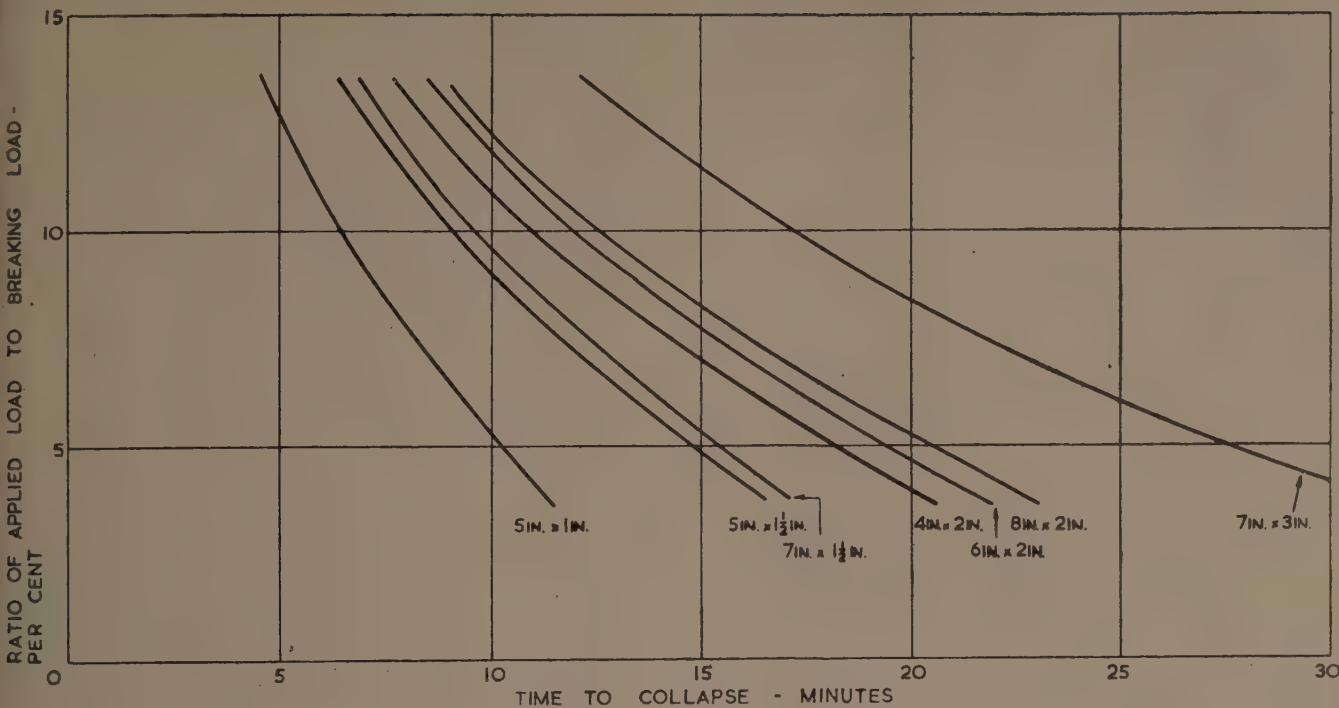


Fig. 8.—The fire endurance of various sizes of timber beams as a function of the load [Crown copyright reserved

loading condition. For any given shape and load the fire endurance will be proportional to the square root of the cross-sectional area. The maximum fire endurance in relation to the cross-sectional area will be achieved for beams of approximately square cross-section, when loaded to a given fraction of its breaking load. This will not result in the greatest timber economy, as the initial strength will be lower than that of beams having a higher ratio of depth to breadth. When this factor is taken into account the fire endurance of a beam is badly affected by its shape.

IX. Acknowledgements

Thanks are due to the Director of Building Research for permission to use, in this report, unpublished results of fire resistance tests planned by Dr. T. W. Parker, of the Building Research Station of the Department of Scientific and Industrial Research, during 1945-6; and to the Director of Forest Products Research, for supplying their stress-graded joists for test.

The computing of the expressions used in this paper was carried out by Miss S. Birtwistle.

The present work was carried out as part of the programme of the Fire Research Board, and is published with the permission of the Director of Fire Research.

References

[1]Fire resistance, incombustibility and non-inflammability of building materials and structures. *British Standards Institution* B.S. No. 476-1932.

[2]Grading rules for structural timber for purposes where the stresses in timber are known. *British Standards Institution* B.S. No. 940 : Part 2 : 1942.

Book Review

Elements of Soil Mechanics in Theory and Practice, by K. Ll. Nash (London : Constable, 1951). 110 pp., 8 in. × 5 in., 9s.

The book starts with a brief but very informative history of soil mechanics which emphasises the all-important fact that although the term "Soil Mechanics" is new, its methods have been known and applied unconsciously by engineers for many years. They have, however, only recently been correlated and standardised sufficiently to form a recognised method of investigation. The history is followed by brief descriptions of the methods used on site investigation and testing, indicating the general principles involved and omitting details which can only concern the more advanced student. Finally, there are very informative descriptions of the application of the results obtained by soil mechanics methods. These descriptions really do demonstrate how soil mechanics can help the practical engineer.

In connection with the passage on site exploration (pp. 14-21) it is considered that more emphasis should be laid upon the importance of careful consideration of the choice of methods used, the extent of the investigation, and its accuracy. It is obvious that all the care in the world over testing soil properties and applying the results to engineering calculations will be futile if they are applied to soils not truly representative of those which are subject to stress and other factors imposed by the work to be undertaken. The fact that the exploration is usually undertaken in places remote from the rigid control of the office or laboratory, makes it all the more important to stress the value of careful and expert consideration.

The author is to be congratulated on the immense amount of useful information which has been condensed into so small a book. The student who aims at a practical engineering career will obtain from this book a very good general idea of the methods and application of soil mechanics. To one who aims at specialising, it is an excellent preparation for the more advanced works. The concise and interesting manner in which the book is written will also make it very acceptable reading for the more experienced engineer who has not had the benefit of an academic training in the subject.

C. B. B.

The Torsional Strength of Structural Members*

By W. B. Dobie, M.Sc., Ph.D., F.R.S.A., A.M.I.C.E., A.M.I.Mech.E.

Introduction

In structural engineering many members are subjected to some torque, although this is only occasionally taken into account. The exact determination of the torsional properties of structural sections is thought to be too complicated for the design office although empirical expressions have been used. For design purposes the torsional properties required are

(a) the stiffness of the member, and

(b) the torque to produce specified stress conditions.

The elementary expression for an elastic bar of constant circular cross-section transmitting a constant torque, T, is

$$\frac{T}{J} = \frac{q}{r} = C\Theta \qquad \text{1}$$

where J is the polar moment of inertia, q is the stress at the radius r, C is the Modulus of Rigidity and Θ is

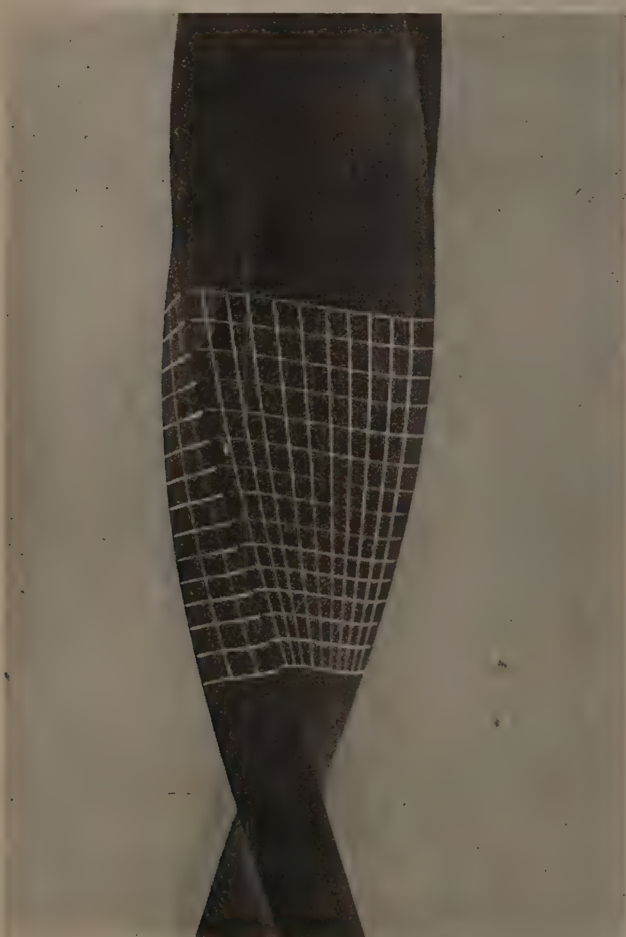


Fig. 1

the angle of twist per unit length. The angle of twist per unit length is constant along the bar length and, writing θ for the maximum twist and l for the length, the stiffness is

$$\frac{T}{\theta} = \frac{CJ}{l} \qquad \text{2a}$$

In terms of the unit twist, the maximum stress is

$$q_{max} = C\Theta R \qquad \text{3a}$$

where R is the maximum radius of the shaft.

Equations 2a and 3a provide the data necessary for the complete design of an elastic member of constant circular cross-section. For non-circular cross-sections the equations are similar but the determination of the parameters corresponding to J and R is more complicated.

The basic assumption, that initially plane cross-sections remain plane on twisting, applies for the circular cross-section but for non-circular cross-sections these planes are distorted ; Fig. 1 shows the distortion in a twisted rubber prism of rectangular cross-section.

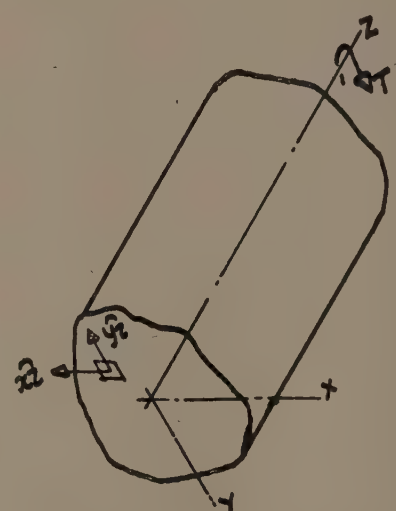


Fig. 2.—Stresses due to free torsion

Because of the warping of the initially plane cross-sections, it is necessary to consider the torsion problem by the mathematical theory of elasticity.

Theory of Elastic Torsion

Considering the equations of equilibrium and the conditions of compatibility, the governing equation to be satisfied at all points in the cross-section is[1]

$$\nabla^2 \phi = -2 \qquad \text{4}$$

where the operator ∇^2 is given by

$$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$$

and ϕ is a stress function ; the stresses, indicated in Fig. 2, are, in terms of the stress function,

$$xz = C\Theta \frac{\partial \phi}{\partial y} \text{ and } yz = -C\Theta \frac{\partial \phi}{\partial x} \qquad \text{5}$$

The boundary conditions are satisfied if

$$\frac{\partial \phi}{\partial s} = 0$$

*Paper to be read before the Institution of Structural Engineers, at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 28th, 1952, at 6 p.m.

where s is the length along the boundary ; since the constant of integration is arbitrary this can be written

$$\varnothing = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 6$$

The torque, T, is given by

$$T = 2\, C\, \Theta \iint \varnothing\ dx\ dy$$

and, rewriting this like equation 1 for the circle, one finds that the property corresponding to J is the torsion constant ,

$$K = 2 \iint \varnothing\ dx\ dy \quad . \quad . \quad . \quad . \quad . \quad . \quad 7$$

and the stiffness for the complex section is

$$\frac{T}{\theta} = \frac{CK}{l} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 2b$$

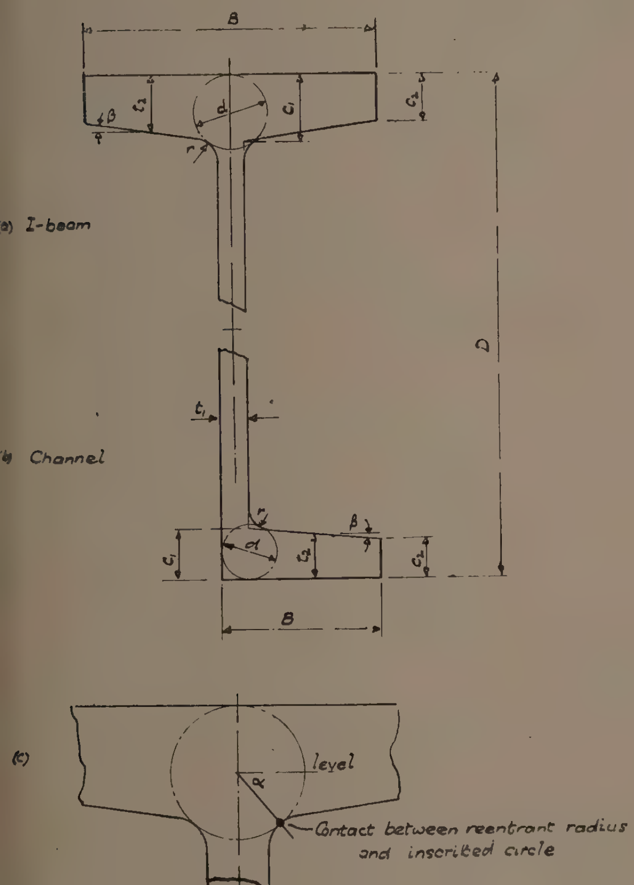


Fig. 3.—Symbols for dimensioning I-beams and channels

The resulting stress, q, at a point in the cross-section, is given by

$$\left(\frac{q}{C\Theta}\right)^2 = \left(\frac{d\varnothing}{dx}\right)^2 + \left(\frac{d\varnothing}{dy}\right)^2$$

so that

$$q = C\,\Theta\ grad\ \varnothing \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 3b$$

Comparison with equation 3a shows that $grad\ \varnothing$ measures the stress and it is usual to denote it by R, the stress factor.

To determine K and R it is necessary to obtain a solution to equations 4 and 6. Solutions can be obtained for structural sections by

(a) analogies, and
(b) numerical computation.

In this investigation the membrane analogy and relaxation methods were used, these being considered the most efficient methods which required the least equipment and which were not laborious. An investigation of the accuracy obtainable by these methods[2] showed that it was of the order of 2 per cent. ; the section examined was circular with a circular groove, which section has a concentration of stress at the bottom of the groove and for which an analytical solution was possible. Both methods of determining the stress function were described in a previous paper[13] to the Institution by Cassie and Dobie.

Torsional Stiffness

Consideration of the Prandtl membrane[4] shows that a complex section, such as an I-beam, can be looked upon as the sum of a number of simpler geometrical shapes with due allowance for the effect of integration into the whole. The obvious division for a flanged cross-section is

$$K = K_w + K_f + K_j$$

where K is the torsion constant of the section,
K_w is the contribution of the web,
K_f is the stiffness of the flanges, and
K_j is the junction effect.

British Standard Beams

The shape of the Prandtl membrane shows that the contribution of the web is approximately equal to that of a rectangle, i.e.,

$$K_w = \tfrac{1}{3}\,(D - 2c_1)\,t_1^3 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 9$$

See Fig. 3 for the interpretation of the symbols.

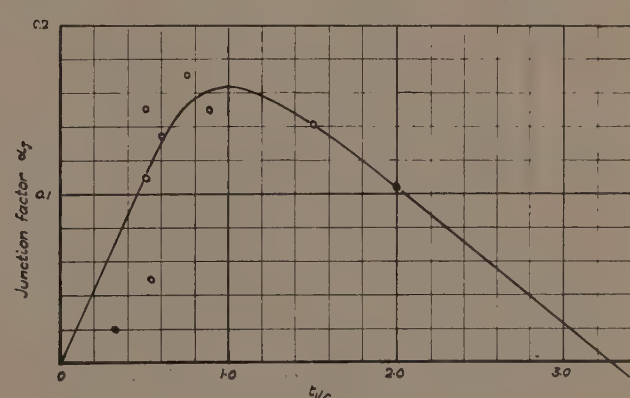


Fig. 4.—Effect of t_1/c_1 on Junction Factor α_j

Tests in which only the flange was modified showed that K_f was a function of $(B - t_1)/t_2$ and t_1/c_1 ; for t_1/c_1 equal to nothing or infinity, K_f was given by the expression for four trapeziums[5], viz.,

$$4K_{tr} = \frac{B - t_1}{6}\,(c_1 + c_2)\,(c_1^2 + c_2^2) - 4V_s c_2^4 \quad . \quad . \quad . \quad 10$$

where V_s is the end constant, normally 0.105. Cassie and Dobie[3] presented the contribution of the flanges as the ordinate of a graph with $(B - t_1)/t_2$ and t_1/c_1 as abscissa and parameter.

Observation of the Prandtl membrane at the flange-web junction showed a hump which was approximately spherical. This suggested[6] the empirical relation for the junctions

$$K_j = 2\alpha_j\, d_0^4 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 11$$

where d_0 was the diameter of the inscribed circle for zero fillet radius and α_j was a junction factor which is given in Fig. 4.

In a similar way, the contribution of the fillets at the re-entrant corner was expressed, for an I-beam, as

$$K_r = 4\alpha_r\, d^4 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 12$$

the radius factor, α_r, being approximately a linear function of r/c_1

$$\alpha_r = 0.0238 \frac{r}{c_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 13$$

and d being the diameter of the largest inscribed circle.

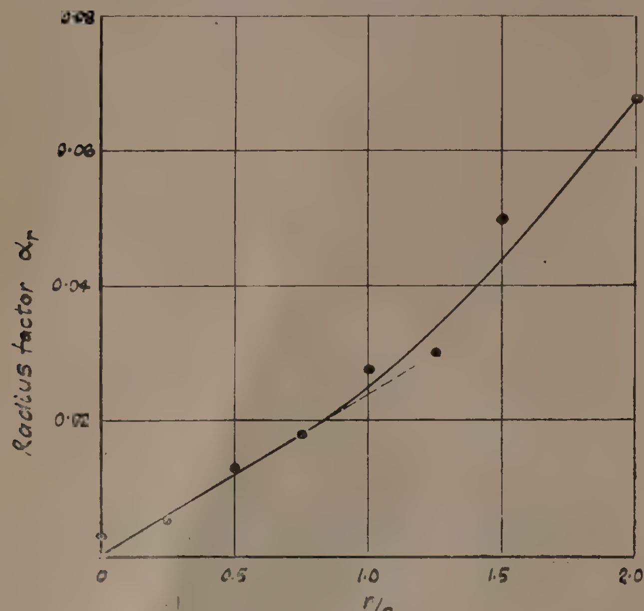


Fig. 5.—Radius factor

The experimental curve for α_r is shown in Fig. 5. An alternative method of calculation is to write

$$K_j + K_r = 2\alpha_j d^4 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 14$$

the comparison between calculated and observed values being shown in Fig. 6. This latter method is sufficiently accurate in the practical range.

Other Structural Sections (Channels, Angles, etc.)

The rules formulated for B.S.B. sections were investigated further before applying to other sections. It was assumed that the components of stiffness K_w, K_j and K_r were the same for all sections and the method of determining K_f was examined more closely.

Fig. 7 shows the dependence of K_f for B.S.B. sections on the ratio t_1/c_1 as well as the obvious term $(B - t_1)/t_2$; this is due to the variations in the flange boundary conditions at the junction with the web. The degree of restraint on the Prandtl membrane at the junction of the flanges and the web determines the increase in K_f above its value K_o when t_1/c_1 is equal to zero or infinity. Since the difference $(K_f - K_o)$, is attributable to the junction it seemed appropriate to express it in a form similar to that used for K_j. Consequently the flange contribution was written

$$K_f = K_o + 2\alpha_{jf}\, d_o^4 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 15$$

and α_{jf}, calculated from Fig. 7, was found to be a linear function of $(B - t_1)/t_2$ as well as a function of t_1/c_1, Fig. 8. The factor α_{jf} was conveniently expressed by the relationship

$$\alpha_{jf} = a\left(1 + 0.28 \frac{(B - t_1)}{t_2}\right) \quad . \quad . \quad . \quad . \quad . \quad 16$$

where the factor a is given by Fig. 9. Equation 16 is applicable only up to $(B - t_1)/t_2 = 12$; at a higher value it is probable that α_{jf} will tend to an asymptote. Further, α_{jf} must be zero for $(B - t_1)/t_2$ equal to nothing, so that equation 16 is valid only for flange finenesses between 2 and 12, which amply covers the practical range.

Also shown in Fig. 9 are the results of a further series of soap film tests (*MAC*). To determine whether the results obtained for B.S.B. sections were applicable to any type of structural shape, the stiffnesses of a number of channel sections were determined: these channel sections were of the following dimensions:

Flange width ...	B	$3.010'' + t_1$
Depth	D	$4.991''$
Flange thickness ...	t_2	$0.995''$
Re-entrant radius ...	r	$0.256''$
Flange taper ...	β	$0°$

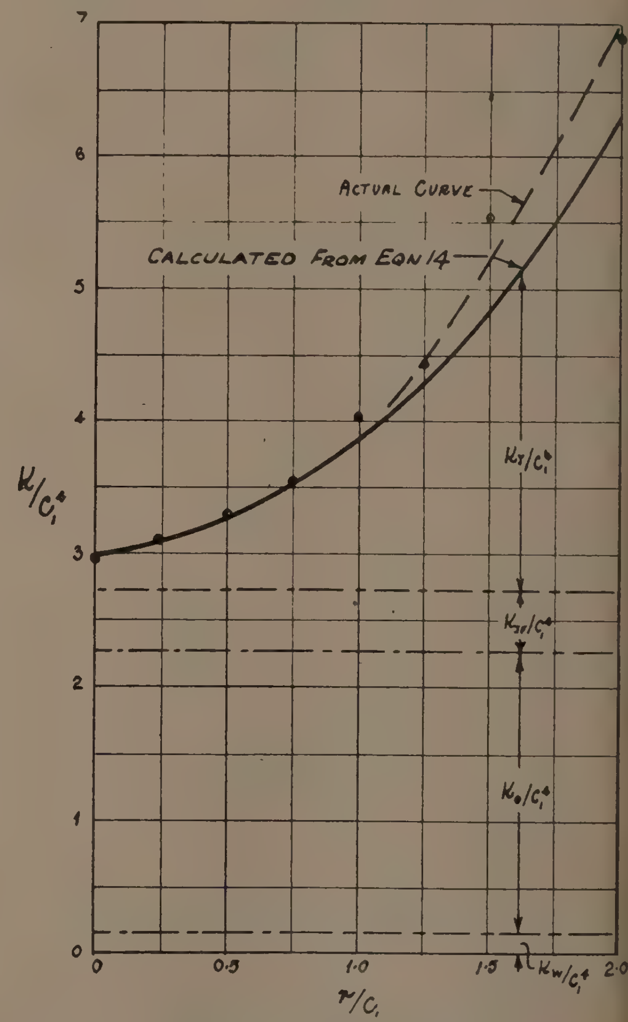


Fig. 6.—Variation of torsion constant K with r/c₁

The web thickness t_1 was continually increased from $0.232''$ to $1.989''$ by $\frac{1}{4}''$ (nominal) steps. The flange contribution was written

$$K_o = {}^2/_3(B - t_1)t_2^3 - 0.42t_2^4$$

At t_1 approximately equal to c_1 a maximum value of $2a$ was obtained, further confirming that this condition produces the stiffest section.

Using Figs. 4, 5 and 9, therefore, the torsional stiffness of most practical solid sections used in structural engineering can be computed. In a recent investigation of the stress distribution in a welded plate girder, Mackey and Brotton[7] measured the torsion constant and found it to be 3.23 in.^{4}; calculated by the method suggested the value was 3.42 in.^{4}, the variation probably being due to a number of causes including slight errors in the method of computing K, and in the experimental results

and possibly due to lack of uniformity at the welded junctions.

Maximum Elastic Stress

Love[8] considers that the strength of a prism to resist torsion depends on the maximum shearing stress and, since the majority of designers use a safe elastic stress as a criterion of design, the clarification of existing data, with particular reference to structural shapes, would eb valuable.

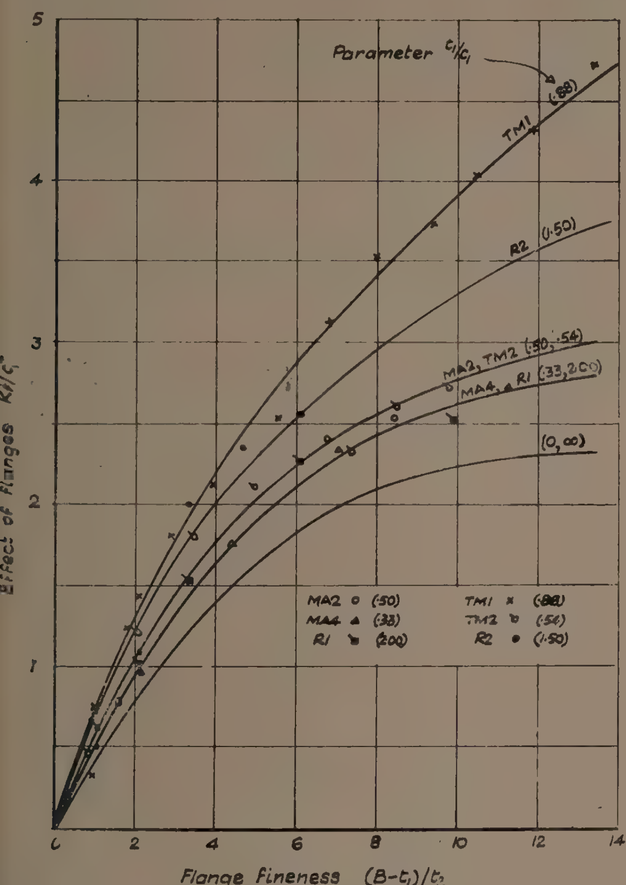


Fig. 7.—Contribution of 8° taper flanges to the stiffness of an I-beam

St. Venant observed in his Memoir[9] that in all the cross-sections enumerated, the points of maximum shearing stress were those points of the contour nearest the centre of the cross-section, but a later statement generalising this was amended in his edition of Navier's "Application de la mecanique."

Boussinesq[10] proved a theorem that the maximum shearing stress occurs at the boundary of a section. In his paper, Filon[11] says that Boussinesq supposed that the "fail" points must be those nearest the axis and showed that this was not necessarily the case. Higgins[12] insists that Boussinesq did not make this supposition and attributes the error in Love's book to Filon's paper. It would appear that the statement, whoever it was originally made by, misled Gibson and Ritchie[13] to suppose that the maximum stress for an I-beam occurred in the web.

Kelvin[14] first pointed out, using the hydrodynamic analogy, that the stress was infinite at re-entrant corners of zero radius. Experience of the Prandtl membrane for technical sections shows that the maximum stress usually occurs at the re-entrant fillet. Griffith[15] found that the maximum stress generally occurred at, or near, the points of contact with the largest inscribed circle, and of these, at the one where the curvature was algebraically the least.

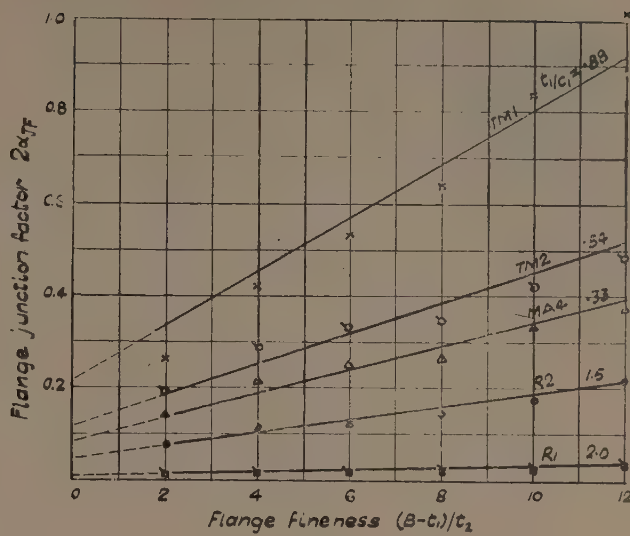


Fig. 8

Analytical solutions for the maximum shearing stress, like the torsion constant, can only be obtained for geometrically simple sections. A useful result, obtained either by a series solution or by consideration of the Prandtl membrane, is that the maximum stress in an infinitely fine rectangle is

$$q_{\max} = C\Theta b = \frac{Tb}{K}$$

where b is the breadth of the rectangle ; the maximum stress occurs at the middle of the longest side.

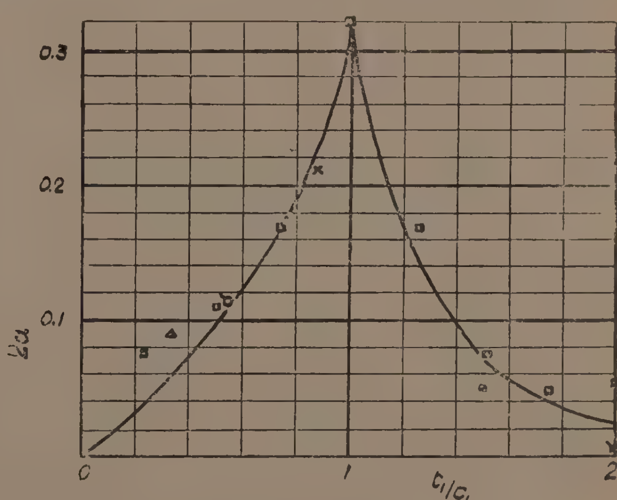


Fig. 9

Practical interest has been centred mainly on the angle with equal leg thicknesses, t. The effect of the re-entrant fillet is normally expressed as a stress-concentration factor using the maximum stress in the leg at a considerable distance from the re-entrant corner as a basis. Fig. 10 shows a number of the results plotted on dimensionless axes. The results of a few previous investigations into the extent of concentrations at the re-entrant corners of I-beams are shown in Fig. 11.

From the published data two conclusions can be formed : first, results obtained by the soap film apparatus show some variation and therefore considerable care is necessary in using the apparatus, and secondly, only the general trend of the maximum stress can be visualised and that only for a specific type of section with equal leg thicknesses.

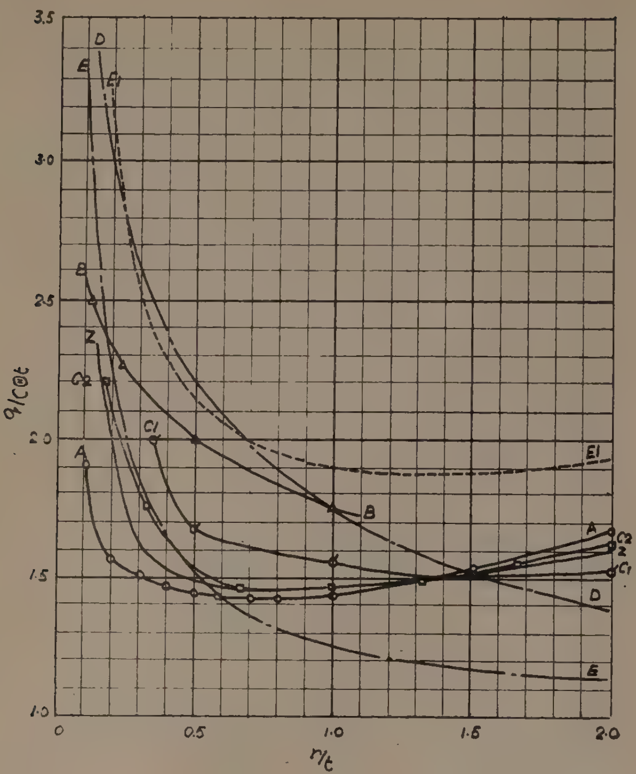


Fig. 10.—Stress concentration at the re-entrant fillet of an equal-leg angle section

A. Griffith and Taylor[23] as a result of soap film tests gave, for concave boundaries

$$\frac{qK}{Td} = \frac{1}{\left\{1 + \left(\frac{\pi d^2}{4A}\right)^2\right\}}\left[1 + \left\{0.118 \log_e\left(1 + \frac{d}{2r}\right) + 0.238\,\frac{d}{2r}\right\}\tanh\frac{2\alpha}{\pi}\right]$$

where d = diameter of inscribed circle.
r = radius of curvature of boundary, and
α = angle turned through by tangent.

In their paper to the Institution of Mechanical Engineers[20] they gave for a re-entrant corner,

$$\frac{qK}{Td} = \frac{1}{\left\{1 + \left(\frac{\pi d^2}{4A}\right)^2\right\}}\left[1 + 0.15\left\{\left(\frac{\pi d^2}{4A}\right)^2 + \frac{d}{2r}\right\}\right]$$

B. Cushman[21] carried out a series of soap film experiments but they were not as extensive as Griffith and Taylor's ; the investigation was mainly concerned with the general use of the membrane analogy.

C. Ehasz, in the discussion[22] of Lyse and Johnston's paper on structural members in torsion gave further experimental results, obtained by the membrane analogy, on two particular angles, 4 in. × 3 in. × ½ in. and 4 in. × 3 in. × ⅜ in.

D. Trefftz[23] used a Schwarz-Christoffel transformation to obtain the maximum stress for a small fillet radius and found that

$$\frac{qK}{Tt} = 1.74\sqrt[3]{\frac{t}{r}}$$

where t was the leg thickness.

E. Timoshenko[1] assumed that the stress function contours in the neighbourhood of the re-entrant corner of an equal angle were parts of circles and that the stress was zero at a distance $\frac{1}{2}t$ from the boundary ; the governing equation then became an ordinary exact differential equation which was integrated to give

$$\frac{qK}{Tt} = 1 + \frac{t}{4r}$$

E1. Assuming the stress is zero at the centre of the inscribed circle one finds that

$$\frac{qK}{Tt} = 1.36 + 0.2\,\frac{r}{t} + 0.34\,\frac{t}{r}$$

The investigation described in the following paragraphs considered geometrically different types of I-beams, the basis of the experimental work being made by relaxation methods and the results checked by the membrane analogy, since its accuracy was found to be reasonable in a controlled experiment.[2]

First, the variables affecting the maximum stress were considered. Previously, the maximum stress was assumed to be a function of the diameter, d, of the largest inscribed circle and the stress-concentration was expressed by a function of a dimensionless parameter involving the re-entrant radius. To specify completely the stress at the re-entrant corner at least as many variables are necessary as are required to specify the shape and size of the section. Such variables would be

$$\frac{r}{d},\ (B - t_1)/t_2,\ (D - 2t_2)/t_1,\ \beta,\ \alpha \text{ and } d.$$

The tentative assumption that the stress can be expressed by a product of the functions of each of the above factors was made and a series of tests designed to verify it. Table 1 gives a summary of the sections examined.

The reasons for carrying out particular series of tests are evident from the details ; series 3 was designed to investigate whether the results obtained from series 1

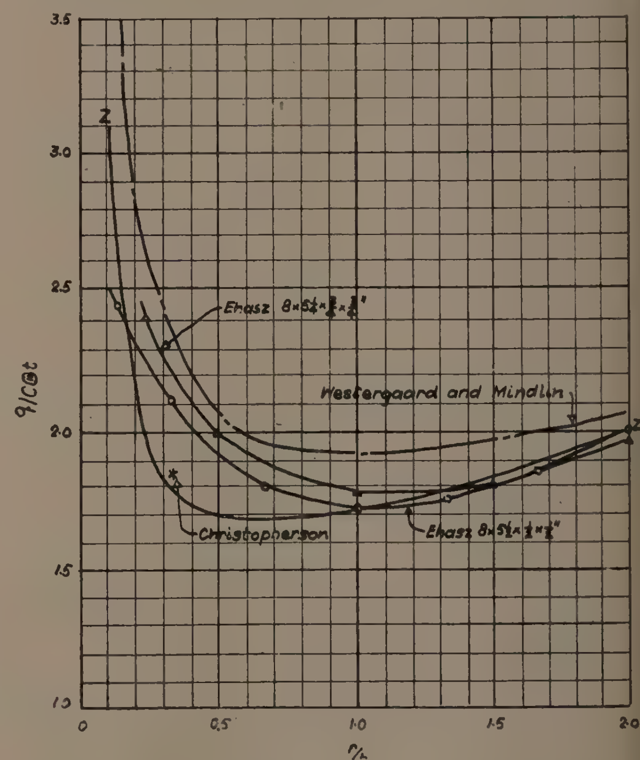


Fig. 11.—Stress concentration at the re-entrant fillet of an I-section with flanges and web of equal thickness, t

Ehasz[22], in addition to the results on angles plotted in Fig. 10, has given some details for two I-beams.

Westergaard and Mindlin[24] obtained a solution for an I-beam by stating that the Prandtl membrane is a surface of constant total curvature and writing this as the sum of the curvature of the contour lines and the transverse curvature of the membrane in the radial direction. On suitable simplifications this yielded

$$\frac{qK}{Tc_1} = \left[1.2 + \frac{1}{3}\left(\frac{C_1}{r} + \frac{r}{c_1}\right)\right]$$

where C_1 was the root thickness of the flanges.

Christopherson[16] in a theoretical investigation of the plastic torsion of a particular I-beam, calculated the elastic stress at the re-entrant corner, by relaxation methods.

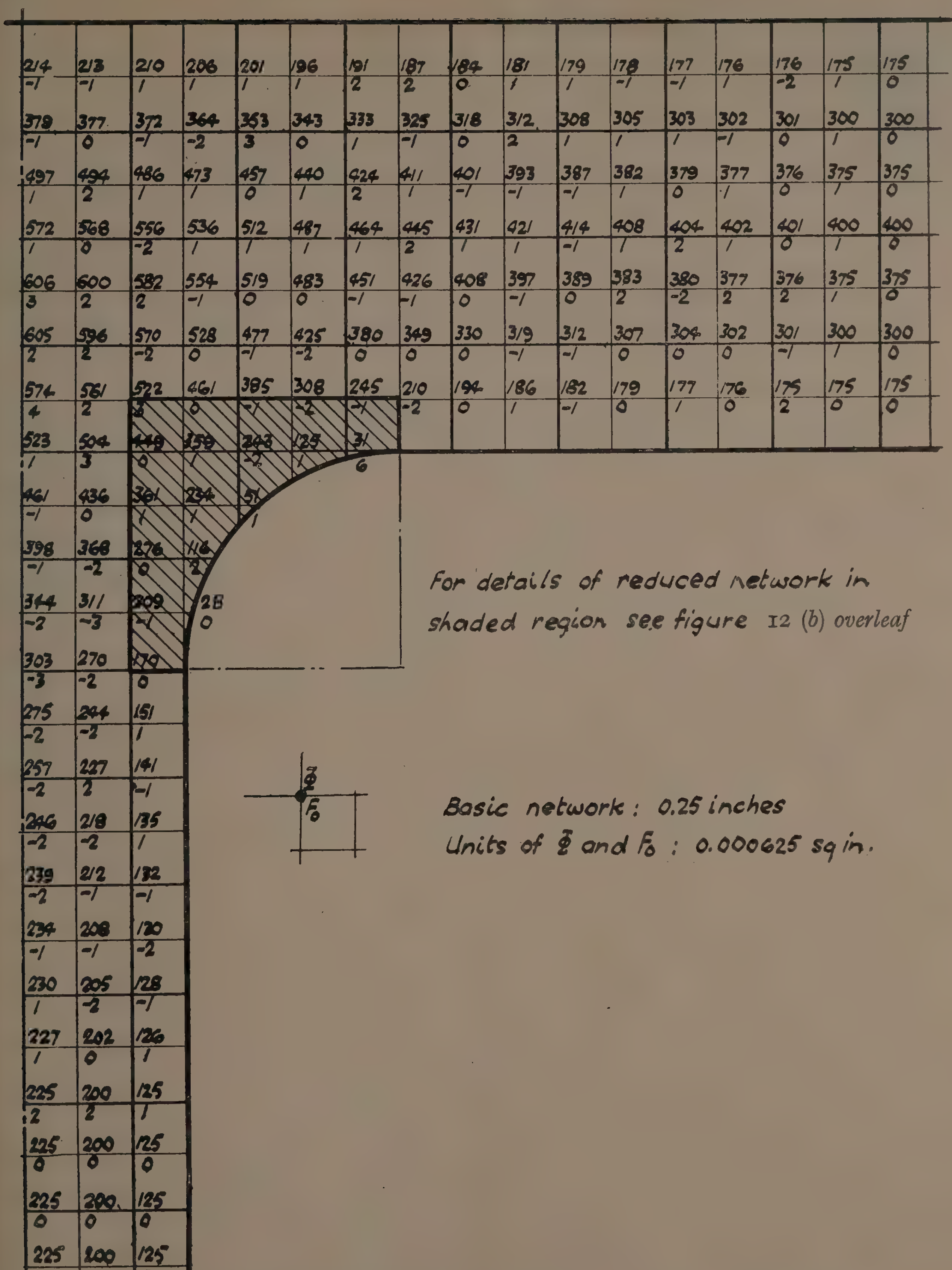


Fig. 12. (a)—Relaxation network for I-beam

and 2, in which only r/d and α were varied respectively, would agree with test results when the two variables were altered simultaneously. If agreement could be obtained it was felt that the tentative assumption expressed previously was valid. The effect of $(D - 2t_2)/t_1$ was assumed to be of the same order as that of $(B - t_1)/t_2$. Only the latter was investigated, series 4, for it was found that the effect was of a very minor nature for B.S. sections, indeed both ratios can be neglected.

The stress function values for the sections marked R in the table were computed by relaxation methods. A basic network of 0.25 was used and in the region of the re-entrant corner this was reduced to 0.0625. Residuals were reduced to 0.0025 or numerically less as shown in the finished network, Fig. 12.

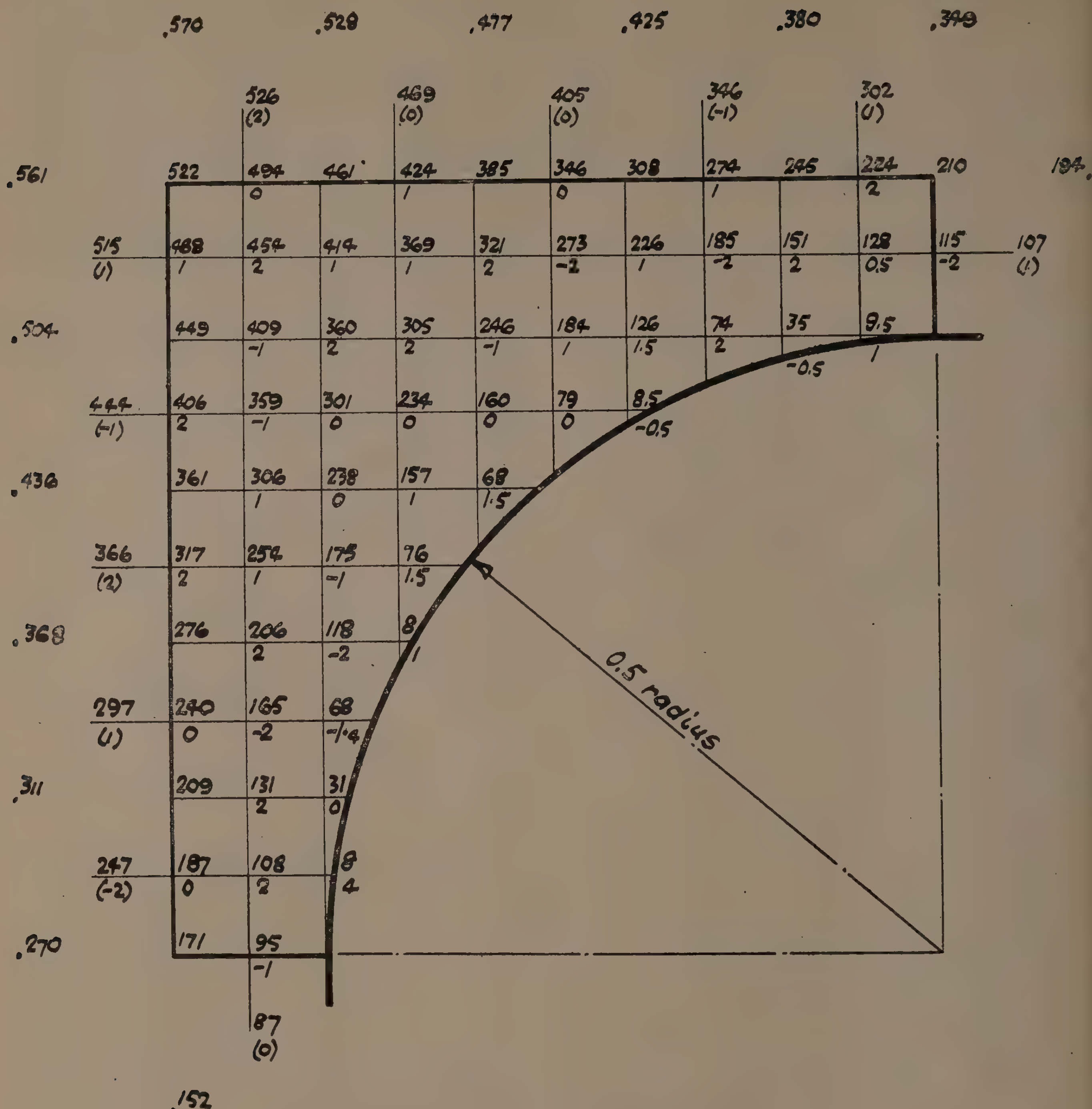


Fig. 12 (b).—Reduced network in region of re-entrant corner

From the stress function values obtained by relaxation methods, the stress was determined in the form $q/C\Theta$ which is a length, R, and which is given by the slope $\partial \varnothing/\partial n$, n being the length of the boundary normal and $\varnothing$ the stress function.

Stresses were determined at the boundary, for points corresponding to each node nearest the boundary, by assuming that the stress function could be represented by a second degree polynomial

$$\varnothing = a_0 + a_1 x + a_2 x^2$$

From Fig. 12c it is obvious that

$$R = \frac{\Phi_1 \dfrac{x_2}{x_1} - \Phi_2 \dfrac{x_1}{x_2}}{x_2 - x_1}$$

Fig. 13 gives three typical stress distributions obtained by this method and shows that the maximum stress occurs in the region of the point of contact with the maximum inscribed circle.

Using the membrane analogy the maximum stress was determined for the sections marked with an M in the table. These results were plotted on the same base as those found by relaxation methods, the test of accuracy of both sets of results being the scatter from the mean line.

In the analysis of the results, dimensionless variables were used, the diameter d expressing the "size" of the section. Fig. 14 shows the effect of r/d and illustrates, as Kelvin first pointed out, that sharp re-entrant corners produce infinite stresses. The effect of the contact angle α, is given in Fig. 15 by the angle factor, $f\gamma$, calculated so that it can be used in conjunction with Fig. 14.

It is interesting to note that when r/d becomes infinite the junction takes the form of an isosceles triangle with

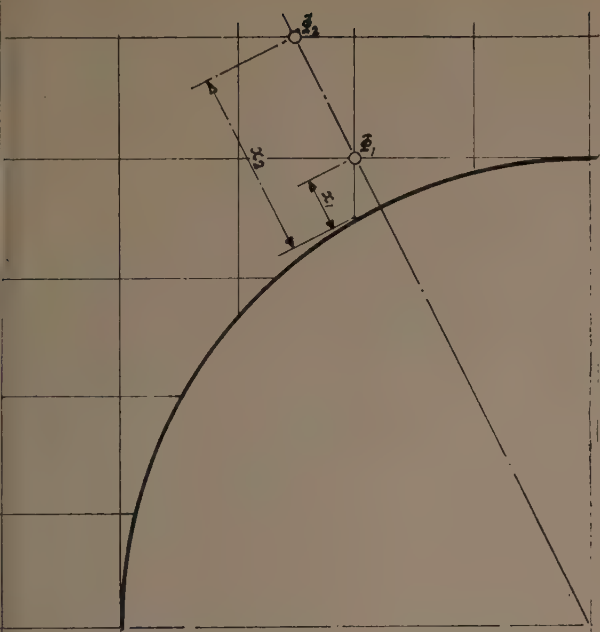


Fig. 12 (c).—Computation of stress at boundary

ase angle of 53°. Using the solution for an equilateral iangle and reducing the base angle to 53° by Fig. 15, ie horizontal asymptote of Fig. 14 is found to be 765, which is in excellent agreement with the experiental results.

To verify the idea of calculating the maximum stress, ries 3 was designed to give simultaneous variations r/d and α. Good agreement is obtained in Fig. 16 tween the experimental values and the curve calcuted from Figs. 14 and 15. For small thicknesses of eb there is some deviation but for practical cases, here t_1/c_1 is usually greater than 0.5, the method ppears to be reasonable.

Figs. 17 and 18 give the results of series 4 and 5, and illustrate the effects of flange-fineness and flange taper. The effect of flange width is given, Fig. 17, as q/q_∞ *vs* $(B - t_1)/t_2$. If t_1, r and β were all zero, the I-section would become two separated rectangles the effect of the fineness of which was calculated. The agreement between the experimental results and the values calculated for the rectangle was so good that no attempt was made to adjust the curve except at small values, at which the effects of finite values of t_1 and r become important. It would seem therefore that the curve has a "tail" which deviates from the curve for the rectangle, the extent of the deviation being a function of t_1/c_1, as drawn. For B.S. sections the flange fineness is always greater than 4 and since t_1 is of the same order as c_1, one might conclude that the effect of flange width can be neglected.

It is possible that there is some compounding of the variables such as the effect of t_1/c_1 on q/q_∞ in Fig. 17. The effect of the flange taper might be a function of variables other than β and in Fig. 15, f_γ may not be independent of r/d. However, in view of the agreement obtained in series 3 such effects have been neglected especially because the sections of interest to structural engineers lie in a small range at the centre of each series.

Application of the curves to compute the maximum stress in structural sections was carried out as follows:

(a) d, r/d and α were calculated :

(b) $q/C\Theta d$ was read from Fig. 14 and f_γ from Fig. 15 :

(c) $q/C\Theta$ was obtained for infinitely fine flanges with zero taper by multiplying together the values obtained in b, and then by the diameter of the largest inscribed circle, d :

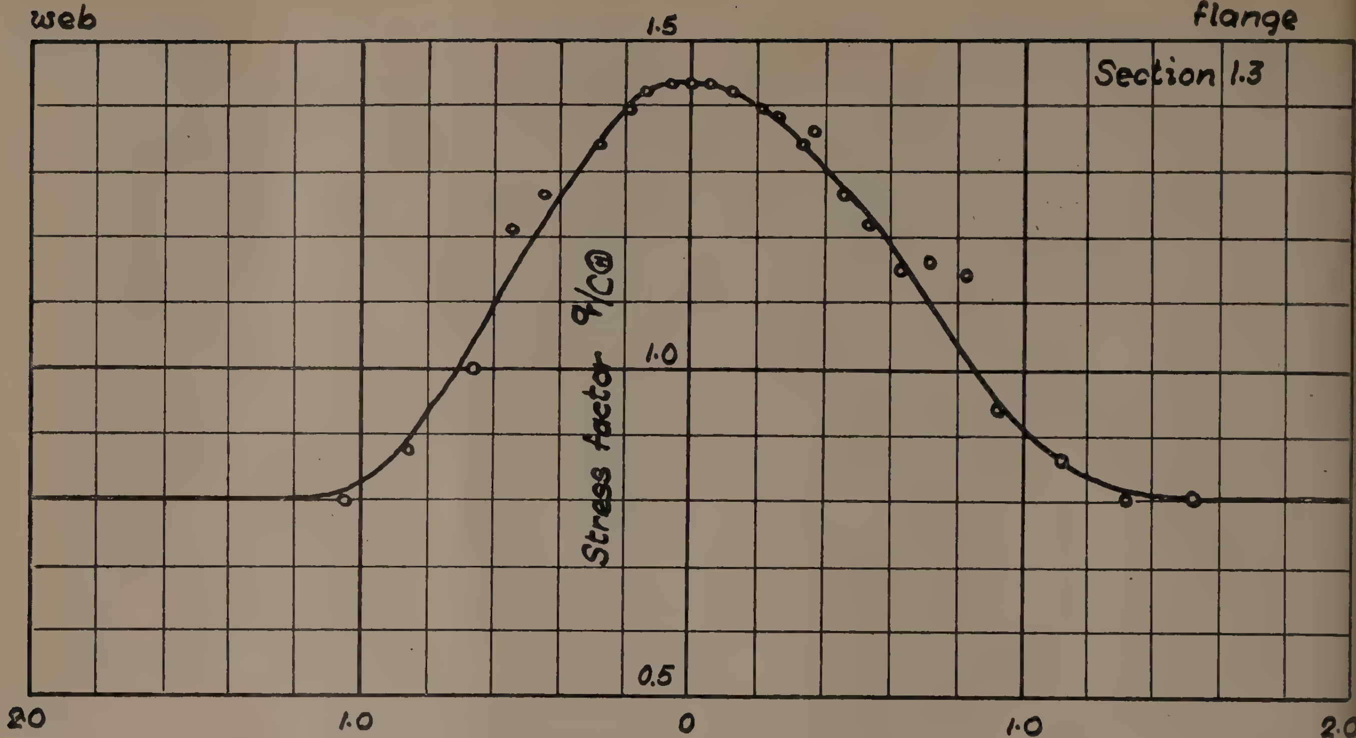
(d) the result of the last calculation was corrected for other variables, using Figs. 17 and 18, where necessary.

In Figs. 10 and 11, the curves marked ZZ were calculated by the method outlined and compare well with the results of previous investigators. In particular

TABLE 1.—Summary of Sections Examined

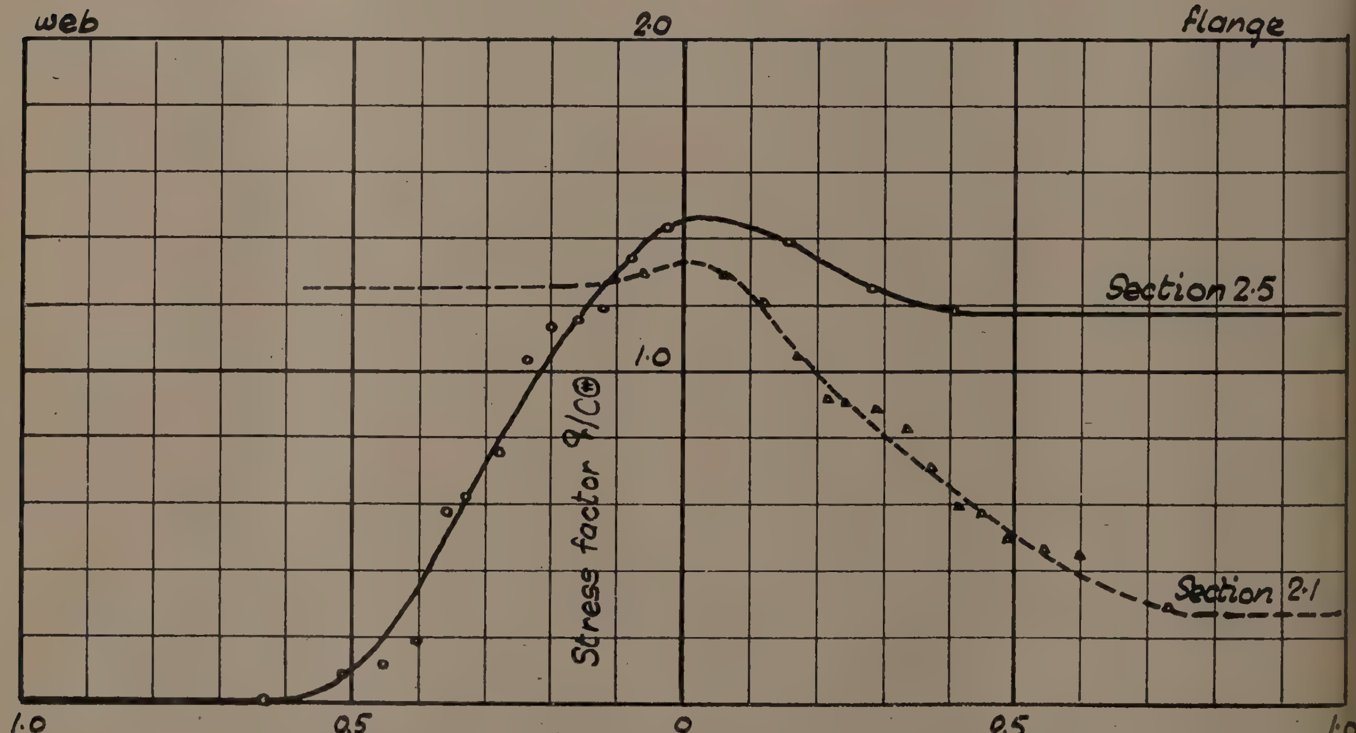
Series	Section	Method of test	d	r	c_1	t_1	$(B-t_1)/t_2$	$(D-2t_2)/t_1$	$\beta°$	$\alpha°$	C_2
	1	*R*	1.500	0.25	1.10	1.10	∞	∞	0	37.0	1.10
	2	*R*		0.50	1.00	1.00					1.00
1	3	*R*		1.00	0.80	0.80					0.80
	4	*M*		1.50	0.60	0.60					0.60
	5	*R*		2.00	0.40	0.40					0.40
	6	*M*		3.00	0	0					0
	1	*R*	1.280	0.50	0.27	1.25	∞	∞	0	6.5	0.27
	2	*M*			0.61	1.06				25.0	0.61
2	3	*R. M.*			1.00	0.50				48.5	1.00
	4	*M*			1.13	0.14				60.0	1.13
	5	*R*			1.17	0				72.0	1.17
	1	*M*	1.125	0.50	1.00	0	∞	∞	0	61.9	1.00
	2	*R. M.*	1.280			0.50				48.5	
3	3	*R*	1.382			0.75				42.8	
	4	*R*	1.500			1.00				37.0	
	5	*R*	2.125			2.00				16.2	
	1	*R*	1.280	0.50	1.00	0.50	1.00	∞	0	48.5	1.00
	2	*R*					1.50				
4	3	*R*					2.00				
	4	*M*					3.50				
	5	*M*					5.00				
	6	*R*					7.00				
	1	*R*	1.280	0.50	1.00	0.50	7.00	∞	0	48.5	1.00
	2	*M*							4.4		0.75
5	3	*R*							9.2		0.50
	4	*M*							13.8		0.25
	5	*R*							18.6		0

R = Relaxation methods *M* = Membrane analogy



Distance along boundary from point of contact with inscribed circle

Fig. 13. (a)—Typical distribution of stress at re-entrant fillet



Distance along boundary from point of contact with inscribed circle

Fig. 13 (b)—Typical distributions of stress at re-entrant fillets

excellent agreement is obtained with the isolated value for an I-beam calculated by Christopherson[16]. Agreement in Fig. 10 indicates that the application of the curves to single-flanged cross-sections gives good results.

Limiting Torque

If one could specify, with accuracy, the load a member was required to carry, the need for factors of safety would depend on the accuracy with which the strength of the member could be computed. Considering a member subjected to pure torsion, professional committees give no guidance on the best methods of design nor do they decide how the strength of a member should be computed. One must therefore use the standard methods of design of bent beams to guide an investigation, and this suggests three possible methods of assessing the limiting torque :

(a) maximum deformation,

(b) limiting stress using the elastic theory, and

(c) simple plastic theory.

Maximum Deformation

The limiting torque depends on the maximum de-ormation agreed upon : if the twist is to be less than θ_m then the limiting torque T_θ is, by equation 2*a*,

$$T_\theta = \frac{CK\theta_m}{l}$$

In the main beams of a structure the torques are often due to the end-fixing moments of secondary members, consequently θ_m for the main member is controlled by the slope allowed in secondary beams. Assuming that for any bent beam the slope could have been limited instead of the deflection, it follows that θ_m should not be greater than the maximum slope. Writing the maximum deflection as 1/350th of the beam span, the maximum slope *s*/350, for the four common cases is

Loading	End Conditions	*s*
Uniformly distributed	free	3.20
	fixed	3.07
Central concentrated	free	3.00
	fixed	3.00

Consequently, θ_m should be less than 1/100 radians and

$$T_\theta = \frac{CK}{100l}$$

It is interesting to compare T_θ with the minimum bending moment, M_δ, required to produce the maximum deflection in a beam of given span, *l*. The minimum bending moment occurs when the beam is bent by two couples, one at each end. Using the simple bending theory, the end-bending moments to produce the maximum deflection of *l*/350, are

$$M_\delta = \frac{EI}{43.7l}$$

and the ratio $$\frac{T_\theta}{M_\delta} = 0.437\frac{CK}{EI}$$

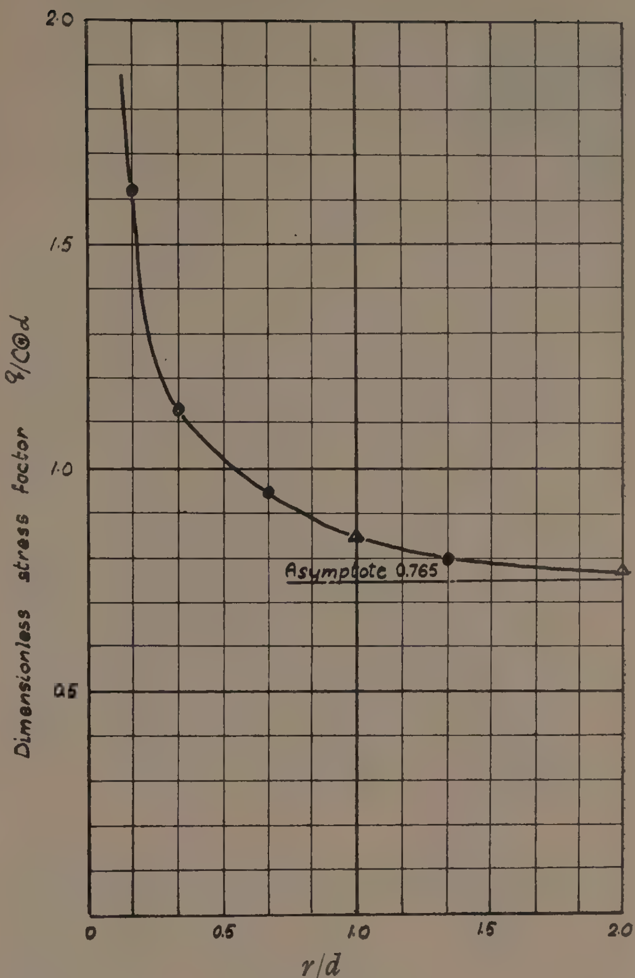


Fig. 14.—Stress concentration at the re-entrant radius of a twisted I-beam

tabulated below for some representative B.S.B. sections, illustrates the inherent torsional weakness of standard sections.

TABLE 2.—Torsional Strengths of Representative B.S.B. Sections

B.S.B.	*K* in.^{4}	$\frac{T_\theta}{M_\delta}$	T_1	T_q	T_p	T_o	T_m
140	8.62	0.000 568	2,620	114,000	193,500	204,000	216,000
135	6.40	0.000 928	2,580	81,000	143,000	142,000	166,200
133	6.89	0.001 180	3,120	101,000	143,000	160,000	179,000
130	2.00	0.000 678	1,000	34,700	72,500	62,500	73,200
129	1.69	0.000 660	821	30,500	66,300	52,500	61,300
128	6.43	0.001 520	3,330	82,500	129,000	144,000	168,000
123	4.23	0.001 880	2,550	58,000	91,400	99,500	114,500
122	2.40	0.001 260	1,420	39,300	71,600	69,000	79,500
108	0.19	0.001 510	228	6,000	12,500	10,100	11,600

Torsional strengths, in inch pounds, were calculated on the following bases :

T_1 Maximum twist limited to 0.01 radians over a beam length equal to 16.5 times its depth.

T_q Material at the point of maximum stress starting to yield, i.e., the maximum elastic stress in the sections equal to the yield stress of 8 tons per sq. in.

T_p Simple plastic theory assumed and the following empirical expression used : $T_p = 0.1415A^{1.8}{}_q$

T_m Mean elastic stress at the boundary of the sections equal to the yield stress, i.e., the stress factor $R = 2A/P$ in equation 3a where A and P are the area and perimeter of the section.

T_o An empirical expression suggested by Orr[19], for which the stress factor, R, is given by :

$$R = \frac{2A}{P} + \frac{1}{6}\left(R_{max} - \frac{2A}{P}\right)$$

To compare the limiting torque calculated by the three methods proposed, the absolute values of T_θ must be established. Since the length, l, of the member does not enter into all three calculations, it was assumed for convenience to be equal to the span of an efficiently designed bent beam.

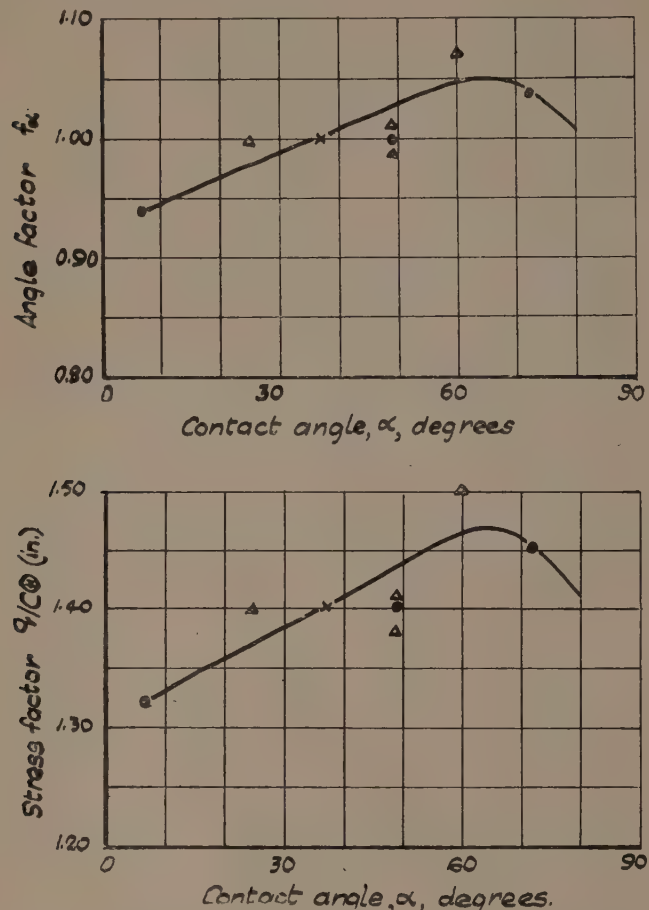


Fig. 15.—Effect of contact angle, α, on the maximum stress in a twisted I-beam

For a beam bent by terminal couples and efficiently designed to deflect the maximum value and be subjected to the maximum elastic stress of 9 tons per sq. in., the span is 16.5 times the depth. Using this beam length, the end torques, T_1, to produce θ_m were calculated, these being considered the limiting torques.

Limiting Elastic Stress

Due to the concentration of stress at the re-entrant corner, plastic flow will start there if the stress exceeds the yield point. Considering a material with a tensile yield point of 16 tons per sq. in., the stress at the re-entrant corner should be limited to 8 tons per sq. in. if the member is to remain elastic. The corresponding limiting torque, T_q, is

$$T_q = \frac{K}{R} \times 8 \text{ tons per sq. in.}$$

which is also tabulated. It is obvious, if one compares T_1 and T_q, that the weakness of B.S.B. sections transmitting a torque, the axial displacements being unrestrained, lies in the deformations produced and not in the induced elastic stresses. Exceptions to the statement could no doubt be produced, e.g., in members where fatigue or stress-corrosion is likely to occur.

If the angle of twist is not limited, the suitability of T_q for determining the limiting torque is in doubt; the high concentration of stress at the re-entrant corner

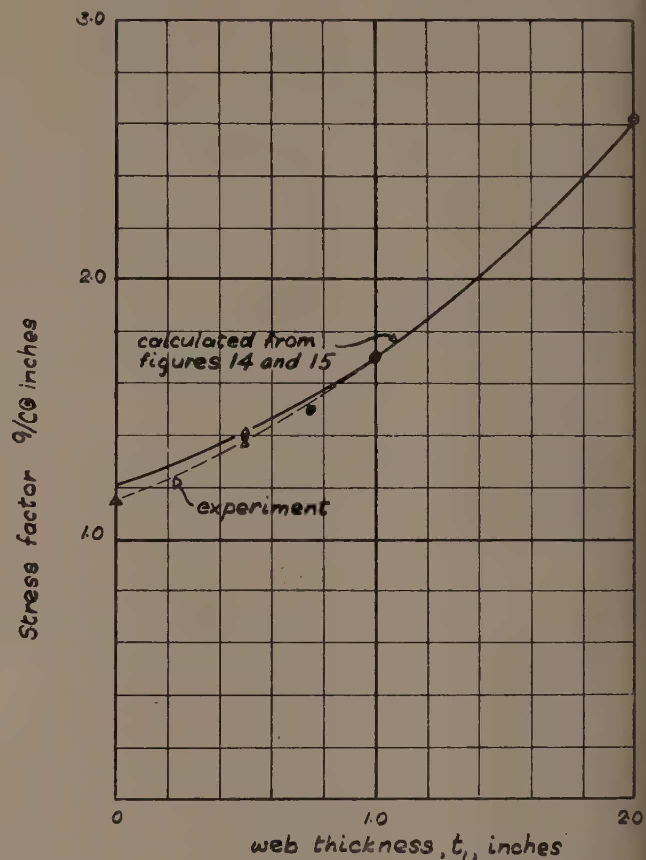


Fig. 16.—Verification of method of calculation of the maximum stress in a twisted I-beam

suggests that it is unsuitable. A better method would seem to be the application of a simple plastic theory.

Simple Plastic Theory

When a bar is twisted so that the stress at certain points exceeds the yield point of the material, a redistribution of stress will occur, the material which yields being relieved of load. If strain-hardening of the material occurs the problem is practically intractable but Christopherson[16] has shown that if it is neglected the results for the limit of plastic overstrain are not erroneous.

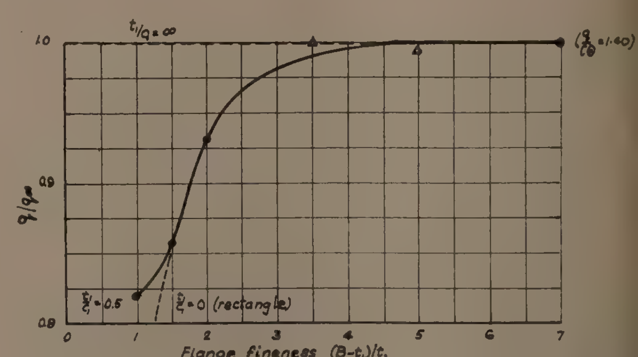


Fig. 17.—Effect of fineness of flanges on maximum stress in a twisted I-beam

Assume then that

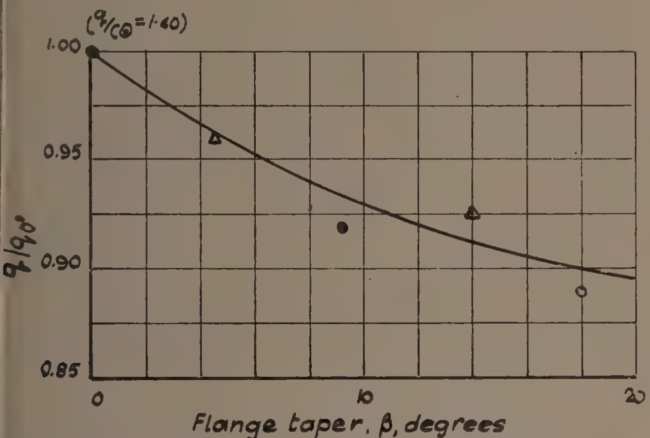
(a) the material of which the beam is made yields according to the maximum shear criterion,

(b) no strain-hardening occurs,

(c) direct and shearing strains are small compared with unity, and

(d) overstraining does not affect the elastic properties of the bar.

In the elastic case the inclination of the Prandtl ıembrane represents the shear stress and the applied ressure is analogous to the torque, for a given surface ension. As the torque is increased the stress at a ›oint in the bar increases until the yield point is reached ; or any further increase in the torque the stress at this ›oint remains constant and equal to the yield point. Considered in the analogy, as the pressure is increased



'ig. 18.—Effect of flange taper on maximum shear stress in a twisted I-beam

he slope of the membrane at a point increases to a naximum which represents the yield stress of the naterial of the bar. From equation 3b the plastic areas of the bar have a stress function represented by a urface of constant slope. Nadai[17] introduced the con-ept of the limiting roof to govern the elevation of the Prandtl membrane. When the bar is twisted to its imit of plasticity, i.e., when it can resist no further orque, the stress function is represented by the Nadai oof.

To get the shape of the limiting roof, Nadai proposed he use of the sand-heap analogy[18] ; dry sand is heaped on a plate of the same shape as the cross-section of the oar and the weight of sand retained on the plate repre-ents the limiting torque. By comparison with the veight of sand retained on a circular section, for which solution is analytically possible, the limiting torque or the plastic section modulus can be determined.

For the rectangle the plastic section modulus is

$$\frac{T}{q} = \left[\tfrac{1}{2}(n - 1) + \tfrac{1}{3}\right] t^3$$

vhere n is the fineness, l/t, and t is the thickness. It is obvious that for geometrically similar bars the section nodulus varies with the cube of any representative limension, or in terms of the area

$$\frac{T}{q} = kA^{3/2}$$

vhere k is a shape factor.

For the rectangle the shape factor, k, is

$$k = \frac{\tfrac{1}{2}(n - 1) + \tfrac{1}{3}}{n^{3/2}}$$

To try to find a simple expression for the limiting torque of structural sections, twenty specimens were tested by the sand-heap analogy ; Fig. 19 shows some specimen sand-heaps. Although it was well-known that B.S. sections are NOT similar, the theory for similar

Fig. 19.—Specimen sand-heaps

bodies was assumed to hold and the results accordingly plotted on a base of area, Fig. 20. The average value of the shape factor was found to be 0.1079 but a better fit was obtained with the empirical line :

$$T/q = 0.1415\ A^{1.3} \text{ (dimensions in inches)}$$

Two sections were also calculated graphically and the results plotted on the graph—good agreement was obtained. Values of the limiting torque, T_P, calculated by the empirical expression are tabulated and show that to produce complete plastification a torque nearly twice that to initiate plastic flow is required.

In addition to the three methods of determining the limiting torque, which have been discussed, two others have been tabulated :

$T_m =$ where the mean elastic stress at the boundary of the section is equal to the yield stress, i.e., the stress factor $R = 2A/P$ in equation 3a where

A and P are the area and perimeter of the section, and

T_o = an empirical expression suggested by Orr[19] for which the stress factor, R, is given by

$$R = \frac{q}{C\Theta} = \frac{2A}{P} + \frac{1}{6}\left(R_{max} - \frac{2A}{P}\right)$$

Neither of these methods produce any specific physical conditions and therefore the simple plastic theory is preferred. It is obvious that T_m might exceed the limiting torque T_p, indeed it might also be greater than the limiting torque determined by experiment. Orr

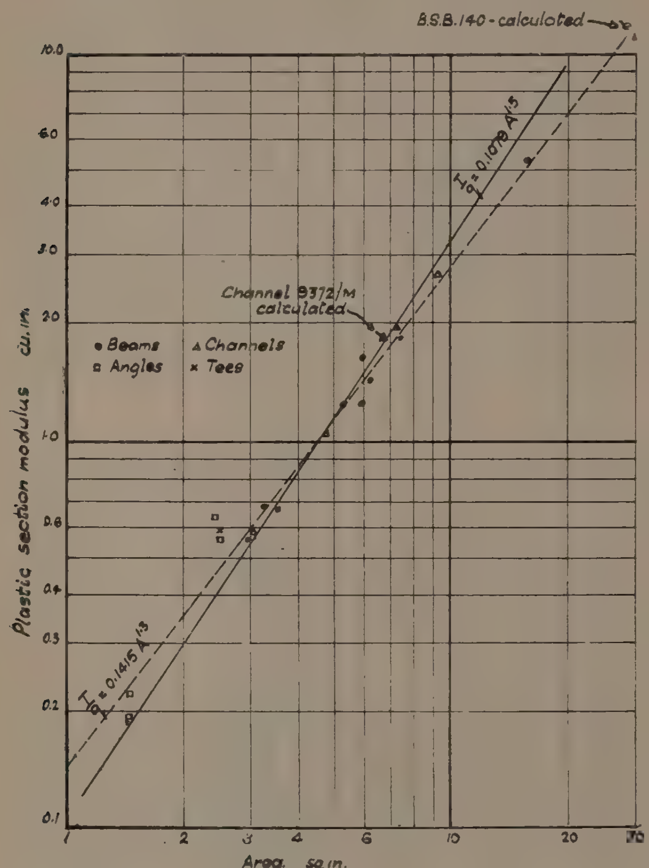


Fig. 20.—Relationship between plastic section modulus and area for structural sections

based his expression on a number of experiments and when one considers the agreement between T_o and T_p it is apparent that T_p is compatible with the practical results and yet retains a simple basic theory. For this reason it appears the most suitable method for determining the limiting torque.

Conclusions

The maximum torque a bar can transmit, irrespective of the deformations produced, is best determined by the simple plastic theory and for structural sections the empirical expression for the limiting torque,

$$T_P = 0.1415\,A^{1.3}q$$

is recommended.

If the design must be based on the elastic theory, a method is described for computing the maximum stress in a structural section. For a material with a tensile yield point of 16 tons per sq. in., the limiting torque should not produce a shear stress greater than 8 tons per sq. in.

In view of the large deformations which occur, the maximum angle of twist should always be checked, indeed it will no doubt be the controlling factor in many cases. The maximum angle of twist should be about 0.01 radians but, in the same way that the maximum deflection and safe stress are specified by the British Standards Institution for members subjected to bending and axial forces, the maximum angle of twist and safe stress should be defined for actions including torsion.

Acknowledgement

The author wishes to acknowledge the encouragement and help given by Professor W. Fisher Cassie and the practical assistance of Mr. A. Gent, B.Sc., Stud.I.C.E.

References

[1]Timoshenko, "Theory of Elasticity," McGraw-Hill, 1st Edition (1934), Chapter 9.

[2]Gent and Dobie, "Accuracy of determination of the elastic torsional properties of non-circular sections using Relaxation Methods and the Membrane Analogy," unpublished paper (1951).

[3]Cassie and Dobie, "The torsional stiffness of structural sections," STRUCTURAL ENGINEER, 26.3 (March, 1948), 154.

[4]Prandtl, "Zur torsion von prismatischen Staben," Phys. Zeit. 4 (1902) 758. See also "Eine neue Darstellung der Torsionspannungen bei prismatischen Staben von beleibigem Querschnitt," Jahrbuch den Deutschen Math. Ver., 19 (1904), 31.

[5]Lyse and Johnston, "Structural members in torsion," Proc. A.S.C.E., 61 (1935), 469.

[6]Trayer and March, "Torsion of members having sections common in aircraft construction," N.A.C.A. Tech. Report No. 334 (1928) 671.

[7]Mackey and Brotton, "An investigation of the stress distribution in a welded plate girder for the Margam plant," STRUCTURAL ENGINEER, 28.2 (February, 1950), 28.

[8]Love, "Mathematical Theory of Elasticity," Cambridge U.P., 4th Edition (1927), 361.

[9]St. Venant, "Memoire sur la torsion des Prismes," Mem. des Savants Etrangers, 14 (1855), 233.

[10]Boussinesq, "Etude nouvelle sur l'equilibre et le mouvement des corps solides elastiques dout certaines dimensions sout tres-petites par rapport a d'autres," J. de Math., 16.2 (1871), 125.

[11]Filon, "On the resistance to torsion of certain forms of shafting with particular reference to keyways," Phil. Trans. Roy. Soc., Series A, 193 (1900), 309.

[12]Higgins, "A comprehensive review of St. Venant's torsion problem," American J. Physics, 10 (1942), 248.

[13]Gibson and Ritchie, "A study of the circular arc bow girder," Constable (1914), 59.

[14]Thomson and Tait, "A treatise on natural philosophy," Oxford U.P., 2 (1867), 699.

[15]Griffith, "The determination of the torsional stiffness and strength of cylindrical bars of any shape," Tech. Report of Advisory Committee for Aeronautics, 3 (1917), 910, R. and M. No. 334.

[16]Christopherson, "A theoretical investigation of plastic torsion in an I-beam," J. App. Mech., 7.1 (March, 1940), A1.

[17]Nadai, "Plasticity," McGraw-Hill, 1st Edition (1931), 135.

[18]Nadai, "Plastic torsion : an experimental determination of the stress distribution in a bar which has been twisted to the limit of plasticity," Trans. A.S.M.E., 53 (1931), 29.

[19]Orr, "Torsional properties of structural and other sections," Proc. I.C.E., Selected Engineering Paper No. 128 (1932).

[20]Griffith and Taylor, "Use of soap films in solving torsion problems," Proc. I.Mech.E. (1917), 755, and Technical Report of Advisory Committee for Aeronautics, 3 (1917), 920, R. and M. (New Series) No. 333. See also reference 15.

[21]Cushman, "Shearing stresses in torsion and bending by membrane analogy," unpublished paper No. 38 of A.S.M.E. (1932).

[22]Ehasz, Proc. A.S.C.E., 61 (1935), 1222.

[23]Trefftz, "Uber die Wirkung einer Abrundung auf die Torsion spannungen in der inneren Ecke," Z.a.M.M., 2 (1922), 263.

[24]Westergaard and Mindlin, Proc. A.S.C.E., 61 (1935), 509.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, December 13th, 1951, at 5.55 p.m., Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership ?

STUDENTS

BOYLAN, Frederick Walter, of Birmingham.
DUDZINSKI, Tadeusz, of London.
LONGRIGG, Thomas Derek, of Nairobi, Kenya.
PENDAL, Bryan James, of Ipswich, Suffolk.
PUGH, Trevor George, of Auckland, New Zealand.

GRADUATES

BUTTERWORTH, Frederick Arthur, of Brierley Hill, Staffs.
CHAPMAN, Kenneth Geoffrey, B.Sc.(Civil), Leeds, of Harrogate, Yorks.
DUGDALE, Aubrey Keith, B.Sc.(Eng.), Manchester, of Normanton, Yorks.
GEE, Jeffrey Ernest Malcolm, of Erdington, Birmingham, 24.
MACDERMID, Eric Campbell, A.M.I.C.E. of Glasgow, W.2.
MORRIS, David Austin, B.Sc.(Eng.), London, of Rustington, Sussex.
NEELAKANTAN, Seshadrinathan, B.E.(Civil) Madras, of Bombay, 19, India.
PAGE, Frank Arthur, B.Sc.(Eng.) London, of Colchester, Essex.
PARTRIDGE, Malcolm Harold, of Walsall, Staffs.
PERRY, Gordon John, of Llanbradach, nr. Cardiff, Glam.
RIMMER, Harold Edwin, of Liverpool.
SINGH, Sandhu Pritam, B.Sc.(Eng.) Glasgow, of Glasgow, W.
STEPHEN, Lawrence Ian, of London,
SUNNAK, Sardari Lal, B.Sc.(Eng.) London, of Liverpool.
TAYLOR, Robert Stanley, of Bexleyheath, Kent.
TOD, William Antony, B.E.(Sydney), of London.
YIH, Raymond, of Glasgow.

ASSOCIATE-MEMBERS

MACFARLANE, George Keith Murray, of Perth, Scotland.
WILBY, Charles Bryan, Ph.D. Leeds, B.Sc.(Civil) Leeds, of Hull, Yorkshire.

MEMBER

CAPLAN, Louis, B.Sc.(Rand), A.M.I.C.E., of Paarden Eiland, South Africa.

TRANSFERS

Students to Graduates

BLACKBURN, Keith Chalkley, of Woodford Green, Essex.
BREWER, John Joseph, of Stafford.
RICHARDS, Gerald William, of Gnosall, Stafford.

Graduates to Associate-Members

BOWEN, Hugh Derrick, of Beckenham, Kent.
CLARK, James Alexander, of Skendleby, nr. Spilsby, Lincs.
EL-KATIB, Mohamed Tarik, B.Sc.(Eng.) Cairo, of London.
GREEN, Harry Whaley, B.Sc.(Eng.) London, of Middlesbrough.
HOLLAND, Ronald Wilber, M.A.(Cantab.), of Stockport, Cheshire.
LESTER, Eric Gerald, of London.
POMFRET, Donald, B.Sc.(Civil) Manchester, of Sidcup, Kent.
ROSS, Donald Ellis John, of Barrow-on-Trent, nr. Derby.
SHAW, Donald Fraser, of Leeds.

Associate-Members to Members

BRINKWORTH, Horace Walter, A.M.I.C.E., of Loughborough, Leicestershire.
CAPPER, Percival Leonard, T.D., M.Sc.(Eng.) London, A.M.I.C.E., of Harrow, Middlesex.
GODMAN, Richard Frederick, A.M.I.C.E, of Longfield, Kent.
GREEN, Frank Montague, of Shirley, Warwickshire.
KING, Jeffrey William Hitchen, M.Sc. Manchester, A.M.I.C.E., of London.
MACLEAY, Douglas Gordon, of Bulawayo, Southern Rhodesia.
WILLIAMS, John Alan, A.M.I.C.E, of Bexley, Kent.

Members to Retired Members

COX, Henry Robert, of East Molesey, Surrey.
LOWE, Albert, of Sale, Cheshire.
MACKMIN, Henry Augustus, F.R.I.C.S., of Coulsdon, Surrey.
MOBBS, Stuart Kilsby, M.I.C.E., of Sutton, Surrey.
PENDLETON, Vivian, Lt.-Cdr., of Southport, Lancs.
PICKERING, Wilfred John, of Epsom, Surrey.
SELWAY, Edward Douglas, F.R.I.B.A., of Bristol.

RE-ADMISSIONS

Associate-Members

PHILLIPS, Thomas William, of Singapore.
VERMA, Ramjee Prasad, of Patna, India.

OBITUARY

The Council regret to announce the deaths of Adam Knowles STEWART, Henry Dale WILLIAMSON (Members) ; Leon John Varney SERRA (Associate-Member).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of Harry Charles DAVIS, Arthur Edward PERKINS, Ernest Albert SCOTT (Members) ; George BRITTON (Retired Member) ; Edmund Taylor FORSTER, George Valentine William SHEPHERD, John T. R. WILDMAN (Associates) ; Norman Pumfret BRAND, Ernest James HICKMAN, John Eaglesfield Cowan HILL, Harold William Roy MAUNDER, Trevor Watts PHILLIPS, Hanamant Harayan RISBUD (Associate-Members) ; Norman Spencer CLEAVER, Ernest DAVIES, Peter John FRETTER, John Reginald GRIMWADE, Ronald Clyde HERON, John Ernest LEVICK, Jack ROBERTSHAW, Frederic Charles SPELDEWINDE, Benjamin WRIGHT (Graduates) ; Andrew Joseph BEBB, William Neil BLACKBURN, Kenneth Arthur William BULTITUDE, John Hugh BUSS, Maurice William COLLINS, Emyr EDWARDS, Jolyon FYFIELD, Geoffrey Thomas HIGGINSON, Raimond Holmes KERLEY, David Granville SHARP, Charles Keith STEPHENS (Students).

EXAMINATIONS—JULY, 1952

The examinations of the Institution will next be held at centres in the United Kingdom and Overseas on July 15th and 16th (Graduateship), and July 17th and 18th (Associate-Membership).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1 :—

Thursday, February 14th, 1952

Ordinary Meeting at 6.0 p.m., when Mr. D. I. Lawson, M.Sc., M.I.E.E., Mr. C. T. Webster, F.R.I.C., and Mr. L. A. Ashton, B.Sc., will give a paper on "The Fire Endurance of Timber Beams and Floors."

Thursday, February 28th, 1952

Ordinary General Meeting at 5.55 p.m. This meeting, which is for the election of members and is open only to corporate members of the Institution, will be followed by an Ordinary Meeting at 6.0 p.m., when Mr. W. B. Dobie, M.Sc., Ph.D., F.R.S.A., A.M.I.C.E., A.M.I. Mech.E., will give a paper on "The Torsional Strength of Structural Members."

Thursday, March 13th, 1952

Ordinary Meeting at 6.0 p.m., when Mr. P. G. Bowie, A.M.I.C.E. (Member), will give a paper on "Faults in Concrete Structures."

Wednesday, March 19th, 1952

Joint Meeting with the Reinforced Concrete Association at 6.0 p.m., when Mr. F. S. Snow, M.I.C.E., M.I.Mech.E. (Past-President), will give a paper on "Recent Industrial Developments at Port Sunlight and Bromborough."

Thursday, March 27th, 1952

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6.0 p.m., when Mr. F. R. Bullen, B.Sc., M.I.C.E. (Member of Council), will give a paper entitled "Unusual Design for a large Constructional Shop."

Thursday, April 24th, 1952

Ordinary General Meeting for the election of members 5.55 p.m., followed by an Ordinary Meeting at 6.0 p.m., when Mr. S. Mackey, M.E., Ph.D., A.M.I.C.E.I. (Associate-Member), and Dr. D. M. Brotton, B.Sc., (Graduate,) will give a paper entitled "An Investigation of the Behaviour of a Riveted Plate Girder under Load."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

JOURNAL CASES AND BINDING, 1951

A binding case can be supplied for the twelve issues of the Journal, January-December, 1951 (Volume 29), price 11s., post free.

The price for binding volumes is 26s. per volume, inclusive. This price is for the half-leather binding which has been in use for some years.

It is requested that all parcels and Journals forwarded for binding should bear the name, address and rank of the member concerned. All volumes for binding must be despatched to the Institution by March 31st, 1952.

An Index will be included in all volumes bound. This Index will not be generally distributed, but members and others wishing to have a copy should apply to the Secretary.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Thursday, February 14th, 1952

Joint Meeting with the Institution of Civil Engineers, North-Western Association, at the Engineers' Club, 17, Albert Square, Manchester, at 6.30 p.m. Mr. Gilbert Roberts, B.Sc., M.I.C.E., on "The Dome of Discovery—Festival of Britain Site."

Monday, February 25th, 1952

Mr. Arthur Bolton, M.Sc. (Graduate), on "A New Approach to the Elastic Analysis of Two Dimensional Rigid Frames," at the College of Technology, Manchester, 6.30 p.m.

Wednesday, March 12th, 1952

Joint Meeting with the Liverpool Engineering Society, at The Temple, 24, Dale Street, Liverpool, at 6.0 p.m. Mr. J. Cunningham, B.Sc., A.M.I.C.E., on "The Britannia Tubular Bridge over the Menai Straits."

Wednesday, March 19th, 1952

Dr. G. G. Meyerhof, M.Sc., A.M.I.C.E., F.G.S. (Associate-Member), on "Some Aspects of Soil Mechanics with Reference to Foundations," at the College of Technology, Manchester, 6.30 p.m.

Tuesday, April 29th, 1952

Professor J. A. L. Matheson, M.B.E., M.Sc., Ph.D., M.I.C.E. (Member), on "Plasticity and Structural Design," at the College of Technology, Manchester, at 6.30 p.m.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 28, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Friday, February 22nd, 1952

Mr. L. E. Hunter, M.Sc.(Eng.), A.M.I.C.E. (Member), on "Moving Form Construction," at The James Watt Memorial Institute, Great Charles Street, Birmingham, at 6.0 p.m.

Monday, February 25th, 1952

Dr. G. G. Meyerhof, M.Sc., A.M.I.C.E., F.G.S. (Associate-Member), on "Some Aspects of Soil Mechanics with reference to Foundations," Merchant Hall, Albion Street, Derby, at 7.0 p.m.

Friday, March 28th, 1952

Mr. P. B. Morrice, B.Sc.(Eng.), on "The Research Station of the Cement and Concrete Association," at Stafford.

Tuesday, April 29th, 1952

Annual General Meeting.

Hon. Secretary : E. R. Deeley, A.M.I.Struct.E., Arranmoor, Adshead Road, Dudley, Worcs.

GRADUATES' AND STUDENTS' SECTION

Wednesday, April 30th, 1952

Mr. S. M. Cooper (Associate-Member) on "Investigation of the Failure of the Tacoma Narrows Bridge." The following films will be shown :—"The Failure of the Tacoma Narrows Bridge" (Long Version) ; "River to Cross" (Wind tests on the Severn Bridge Model.) The meeting will be held in the James Watt Memorial Institute, Great Charles Street, Birmingham, at 7.0 p.m.

NORTHERN COUNTIES' BRANCH

The following meetings have been arranged :—

Tuesday, February 5th, 1952

Mr. L. Scott White, O.B.E., M.I.C.E. (Past-President), on "The Moving of King Henry VIII's Wine Cellar, Whitehall Gardens," at the Cleveland Scientific and Technical Institution, Middlesbrough.

Wednesday, February 6th, 1952

The above meeting will be repeated at the Neville Hall, Newcastle.

Tuesday, March 4th, 1952

Mr. G. S. Gowland (Associate-Member), on "Impressions of U.S.A. Welding Methods," at Middlesbrough.

Wednesday, March 5th, 1952

The above meeting will be repeated at Newcastle.

Wednesday, April 2nd, 1952

Annual General Meeting at Newcastle, followed by a paper entitled, "Data in the Drawing Office," by Mr. J. Ross.

All meetings will commence at 6.30 p.m., preceded by tea at 6.0 p.m.

Hon. Secretary : Ian MacGregor, M.I.Struct.E., 9, Ellison Place, Newcastle upon Tyne, 1.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, February 5th, 1952

Mr. R. Montgomery (Associate-Member), on "Some Economies in the Fabrication of Steel Sections," at the College of Technology, Belfast, 7.30 p.m.

Tuesday, March 4th, 1952

Mr. H. M. Nelson, B.Sc., A.R.T.C., on "Plastic Design Applied to Structural Engineering," at the College of Technology, Belfast, 7.30 p.m.

Tuesday, April 22nd, 1952

Annual General Meeting.

The above meetings will be held at the College of Technology, Belfast, at 7.30 p.m.

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Wednesday, February 13th, 1952

Mr. J. Dixon and Mr. J. M. Campbell, B.Sc. on "Site Exploration and Rock Drilling Methods."

Tuesday, March 11th, 1952

Mr. W. A. Fairhurst (Member), on "The Design of Engineering Structures, including Concrete Bridges."

Thursday, April 17th, 1952

Annual General Meeting.

The above meetings will be held at the Ca'doro Restaurant, Union Street, Glasgow, at 6.0 p.m.

Hon. Secretary : D. G. Drummond, B.Sc., A.M.I.C.E., M.I.Struct.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES' BRANCH

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10/12, Market Street, Torquay.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, February 13th, 1952

Mr. Wallace A. Evans (Member), on "The Completed Abertillery Bridge," at the Mackworth Hotel, Swansea, 6.30 p.m.

Friday, February 15th, 1952

A meeting will be held at Colwyn Bay, details of which will be announced later.

Tuesday, February 19th, 1952

Mr. Wallace A. Evans (Member), on "The Completed Abertillery Bridge," at the South Wales Institute of Engineers, Cardiff, 6.30 p.m.

Wednesday, March 5th, 1952

A meeting will be held at Swansea, when the films referred to above will be repeated.

Friday, March 21st, 1952

A meeting will be held at Colwyn Bay, details of which will be announced later.

Tuesday, April 1st, 1952

Students' Evening.

Friday, April 25th, 1952

Annual Dinner at Osborne Hotel, Swansea.

Hon. Secretary : E. R. Steward, A.M.I.Struct.E., Edrom, Ashleigh Road, Blackpill, Swansea.

WESTERN COUNTIES' BRANCH

The following meetings have been arranged :—

Friday, February 1st, 1952

Combined meeting with the Institution of Civil Engineers. Dr. A. R. Collins, M.B.E., A.M.I.C.E. (Associate-Member), on "Some Effects of Recent Developments on the Design and Construction of Concrete Structures."

Wednesday, February 20th, 1952

Annual Dinner at the Royal Hotel, Bristol.

Friday, March 7th, 1952

Mr. J. A. Newton (Student), on "The Civil and Structural Engineers' Contribution towards the Reconstruction of Stapleton Road Gas Works, Bristol."

Friday, April 4th, 1952

Annual General Meeting, followed by Film Show.

Hon. Secretary : C. E. Saunders, M.I.Struct.E., Dunkery, Edward Road, Walton St. Mary, Clevedon, Somerset.

YORKSHIRE BRANCH

A meeting of the Branch was held at the Great Northern Hotel, Leeds, on November 21st, 1951, and was attended by 48 members and visitors. Mr. T. E. S. White, B.Sc., M.I.C.E., gave a paper on "The Baitings Reservoir for Wakefield Corporation," which was illustrated by a film taken by the author. The paper was followed by an interesting discussion, at the conclusion of which a vote of thanks to the speaker was proposed by Mr. W. Hunter Rose, and seconded by Mr. S. Mackey.

The following meetings have been arranged :—

Friday, February 8th, 1952

Annual Dinner and Dance, Parkway Hotel, Otley Road, Leeds, 7.0 p.m.

Wednesday, February 20th, 1952

Combined meeting with the Yorkshire Association of the Institution of Civil Engineers. Mr. James N. Garden, A.M.I.C.E., on "Prestressed Concrete Bridge, Skelton Grange Power Station, Leeds," 6.30 p.m.

Wednesday, March 5th, 1952

Mr. S. Champion, M.Sc., Ph.D., A.M.I.C.E. (Associate-Member), on "Deep Mining Shafts," at the Technical College, St. George Gate, Doncaster, 7 p.m.

Wednesday, March 19th, 1952

Mr. Hugh B. Sutherland, S.M.(Harvard), A.M.I.C.E. (Associate-Member), on "Problems in Foundation Engineering," at the Great Northern Hotel, Leeds, 6.30 p.m.

Wednesday, April 23rd, 1952

Annual General Meeting, to be followed by a lecture to be arranged.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days, Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

ADDITIONS TO THE LIBRARY

Aluminium Development Association. *Symposium on Aluminium in Road Transport.* London, 1951.

American Institute of Steel Construction. *Manual of Steel Construction.* New York, 1950. Presented by Mr. G. W. Harris.

Association of Consulting Engineers. *Festival of Britain, 1951 : Some Examples of Britain's Contribution to Engineering.* London, 1951. Presented by Mr. F. S. Snow.

British Standards Institution Year Book (London, 1951).

CHATE, R. V. (Editor). *The Reinforced Concrete Review,* Vol. I, 1945-49. London, 1951. Presented by Mr. R. V. Chate.

CLARK, J. G. (Editor). *Welded Deck Highway Bridges.* Cleveland, Ohio. 1950. Presented by Mr. H. A. Cadwell.

DU-PLAT-TAYLOR, F. M. *The Design, Construction and Maintenance of Docks, Wharves and Piers.* 3rd Edition. London, 1949. Presented by Mr. F. W. Davenport.

GRASSIE, J. C. *Elementary Theory of Structures.* London, 1950. Presented by Dr. A. A. Fordham.

HARRIS, C. O. *Elementary Structural Design.* Chicago and London, 1951. Presented by Mr. D. W. Cooper.

JENNINGS, J. *Mathematical Solution of Engineering Problems.* London, 1951. Presented by Professor W. T. Marshall.

LAING, John & Son. *Team Work : The Story of John Laing & Son.* London, 1951. Presented by the Publishers.

MAZZONI, A. *The Steam Vents of Tuscany and the Larderello Plant.* Bologna, 1948. Presented by the Author.

National House-Builders Registration Council. *Specification for Houses and Flats.* London, 1949.

PEATFIELD, A. E. *Engineering Components and Materials (Teach Yourself Mechanical Engineering,* Vol. II). London, 1951. Presented by Mr. W. Morgan.

SALIGER, R. *Fortschritte im Stahlbeton durch hochwertige Werkstoffe und neue Forschungen.* Vienna, 1950. Presented by Mr. P. M. Tezner.

SCHENK, W. *Der Rammpfahl* (Berlin : Wm. Ernst, 1951). Presented by Dr. G. G. Meyerhof.

TAYLOR, F. Johnstone. *Modern Bridge Construction.* (London, 1951.) Presented by Mr. R. P. Mears.

VAWTER, J. and CLARK, J. G. *Elementary Theory and Design of Flexual Members.* New York and London, 1950. Presented by Mr. T. J. Reynolds.

WYNN, A. E. *Design and Construction of Formwork for Concrete Structures.* 4th Edition. London, 1951. Presented by Mr. D. J. Bennett.

REPRESENTATION

The Council have made the following nominations o members to represent the Institution :—

ROYAL SANITARY INSTITUTE CONGRESS, MARGATE, 1952

Mr. J. E. Swindlehurst, O.B.E., M.A., M.I.C.E. (Past President).

BRITISH STANDARDS INSTITUTION COMMITTEE USM 2. UNITS, ABBREVIATIONS AND SYMBOLS

Colonel J. C. P. Tosh, M.C. (Technical Officer of the Institution).

AD HOC COMMITTEE FOR STRUCTURAL SAFETY

The following Ad Hoc Committee of the Science and Research Committee has been constituted :—

AD HOC COMMITTEE FOR STRUCTURAL SAFETY

The President (*ex officio*)
Professor A. L. L. Baker
Mr. J. Guthrie Brown
Lt.-Col. G. W. Kirkland
Professor A. G. Pugsley
Dr. F. G. Thomas
Mr. W. E. Thorowgood
Mr. Leslie Turner
Mr. H. A. Cadwell
Mr. H. V. Hill.

TECHNICAL REPORTS OF THE INSTITUTION

10 (II) 1946 FORMULAE FOR COMPUTATION OF STRESSES

This Report is being reprinted and will shortly be on sale, price 3s. 6d. per copy. (Copies of the graphs in the Report will be available in separate sheets for office use ; price 5s. per complete set of graphs.)

25 (I) 1942 REPORT ON FOUNDATIONS—PART I. FOUNDATIONS IN DISTURBED GROUND

The above Report, stocks of which are now exhausted, is withdrawn from sale.

DRURY MEDAL COMPETITION, 1951

The Council have awarded the Drury Medal for 1951 to Mr. Ronald Frank DAVIDSON (Graduate), of Manchester. The problem set for this, the third Competition, was the design of a reinforced concrete garage with provision for erection above it of future superstructure, the garage being subjected to unbalanced earth pressure.

BUILDING RESEARCH CONGRESS, 1951

COPIES OF PAPERS PRESENTED

Books containing the papers presented in the three Divisions of the Congress are obtainable, price 22s. 6d. each or 50s. for the set of three, from the Organising Secretary, Building Research Congress, 1951, Building Research Station, Garston, Watford, Herts., and early application is desirable.

Members of the Institution of Structural Engineers may obtain single books at 17s. 6d. each on application to the Institution, enclosing remittance ; cheques should be made payable to " Building Research Congress, 1951."

The *Division 1* book contains papers on building techniques, structural matters, and soil mechanics.

The *Division 2* book contains papers on building materials.

The *Division 3* book deals with acoustics, heating and ventilating, lighting, and with problems of hospitals, factories and schools.

The three books together present an up-to-date picture of the present position of building research.

The record of discussion which took place at the technical sessions of the Congress may be ordered now in advance of publication from the Organising Secretary at the Building Research Station, price 25s. (20s. to Congress members).

Faults in Concrete Structures*

By P. G. Bowie, A.M.I.C.E. (Member)

Summary

An examination of a number of reinforced concrete ructures erected during the past 50 years indicates that, hile normal practice in design provides an adequate serve of strength and sufficient durability within a uilding, there is reason to think that some modifications these standards are desirable where the structure as a hole, or in part, is exposed to the weather.

The qualities essential to an engineering structure are rength and durability. As to strength, perhaps the est evidence is the continued use of buildings erected the 1900-1910 period. Internally few defects are

Fig. 1.—Bomb damage

pparent, the columns are, by modern standards, rather ose together, the shuttering obviously not quite all at could be desired, while the slabs are often thin and ightly dished. Whether the latter defect is due to eep, displacement of steel at the supports or bond slip duced by vibration, is difficult to say.

Of more recent structures, the fact that it has been ssible to repair and re-use buildings which were gravely amaged by bombing is evidence of both sound design d adequate strength.

During the whole time there has been little change basic design. The recommendations of 1907 and 48 are very similar. Rules for the design of flat abs and for bending in columns have been added, the tter being offset to some extent by taking into account e whole area of the column instead of the core alone, e increased stresses on the concrete being fairly mparable with the wider understanding of its proper- es and manufacture.

It is probably easier to calculate the moments induced in a theoretically monolithic framework than to arrange the reinforcement in such a way that excessive bond stresses do not occur at the junction, particularly in the case of external columns. High bond stresses may lead to a reduction in the bending moment on the column, but as in continuous beams, if a re-distribution of moments is assumed, it is advisable to consider whether "the marked increase in crack widths is not important."

Inside buildings, the most obvious faults occur in applied floor finishes, that is, in toppings which for one reason or another are laid after a suspended floor has hardened. The bond between the two has to resist not only the tendency of the topping to lift at the edges as it dries, but also the horizontal shear at the supports where, in addition, the finish itself is subject to quite high tensile stresses owing to the greater distance of the top surface from the neutral axis. One difficulty is to decide how far such a finish can be counted upon as being part of the slab. If it is thought justifiable to include half the thickness of the topping in the calculations, it must then be considered whether the bent-up bars lying in the plane between the two classes of concrete are happily situated. In terrazzo work, especially at stair landings where there are additional stresses due to the expansion or contraction of the building as a whole, cracks of this kind are very liable to

Fig. 2.—Limonite stains

occur unless dividing strips have been placed near the supports.

No material of an alkaline character can be expected to resist attack by acids, whether due to spillage from industrial processes or as a result of acidic gases being absorbed by the moisture on a concrete surface, such as the ceiling over a boiler, and probably some positive protection is desirable.

On the whole, however, there is ample evidence that reinforced concrete as designed in the past has a very

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**Paper to be read before the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1, on Thursday, March th, 1952, at 6 p.m.*

satisfactory reserve of strength, and that under normal conditions inside a building it is virtually permanent.

Externally, a concrete structure is subject to far more extreme conditions, the range of temperature is greater, and moisture is generally present, with a greater consequent liability to corrosion of reinforcement.

Permanence is perhaps too absolute a term to apply to any structure. Durability which, to paraphrase

Fig. 3.—Frost cracks

CP 3, may be said to be the quality of maintaining satisfactory appearance and performance without incurring excessive maintenance or repair, is more reasonable.

From this point of view it may be desirable to consider whether, as with some other materials, present practice should be modified according to exposure.

The durability of reinforced concrete is affected by factors of a chemical nature due to its composition or surroundings, physical considerations such as frost or heat, and factors connected primarily with the detail design of the structure.

There is no evidence in this country of any aggregate reaction when supplies are drawn from natural sources, except in the case of a very rare type of sandstone. A few dolerites contain veins of chlorophæite, which, by expansion when oxidised, may result in the formation of longitudinal cracks. The objectionable brown surface stains sometimes seen on a concrete surface may of course be due to small pieces of fixing wire or nails left in the shuttering, but are often due to small pieces of soft limonite being present in the gravel aggregate.

The care now given to proportioning in the manufacture of cement rules out any chance of "blowing" due to lime being present in an unsuitable form, for which the Chatelier and pat tests were introduced.

Impurities in the mixing water were found by Professor Duff Abrams to have little effect on the strength of concrete, except in the case of sugar and such related compounds as alcohol and fruit juices.

Weakening of a hardened concrete may be the result of direct attack by acids which are comparatively easy to identify. The presence of a white slimy substance in the concrete surface is generally an indication of what one might call an exchange reaction between the cement and any sulphate solution in contact with it. Deterioration may be limited to some slight surface effect, but if, owing to evaporation from some other part of the surface, the solution is drawn continuously into the body of the concrete by a kind of wick action, the exchange continues and the cement becomes weak and porous. Examples include sewer pipes laid in trenches passing through sulphate-bearing clays, and retaining walls with industrial waste filling behind them.

In seawork something of a similar kind may happen between wind and water, accelerated perhaps by the crystallisation of salts within the surface, but a good quality dense concrete, not subject to abrasion or appreciable tensile stress, does not appear to be greatly affected. Nevertheless, in cases of this kind it may be advisable to take advantage of the greater resistance offered by certain cements in which the more sensitive component tri-calcium aluminate is minimised.

Prominence of aggregate particles on a concrete surface is largely due to the solution in rain-water of the 10 to 15 per cent. of free lime which a set cement contains, a process which is rather uncomfortably named "lixiviation."

The stalactites sometimes seen below bridge floors, especially at cracks or construction joints, and the encrustation at the base of flat-topped masonry walls laid in poor mortar, are further examples of this process. The obvious remedies in such cases are to waterproof the bridge floor and slope the top of a wall or other projection up to as much as 45° so that the water may not soak into the concrete.

This leaching process is most harmful if the flow of water is continuous; even good concrete is seldom absolutely damp or gas tight, mainly due to the use of nearly twice as much mixing water as is needed for the hydration of the cement, so that when concrete is placed by hand in the shuttering, moisture rises in the mass and the upper part may be more porous than the concrete lower down.

Fig. 4.—Fine grading

Any but the densest concrete may become saturated with water, and then if frozen the expansion of the water leads to the formation of cracks usually parallel with the colder surface, and to ultimate disintegration of the concrete. The actual mechanics of this bursting process is still in dispute.

One contention is that moisture rising in the concrete when it is cast tends to collect beneath the particles of aggregate. When the concrete subsequently freezes

this moisture expanding creates tension across planes parallel to the surface.

Another suggestion is that as the outer surface becomes sealed by frost the water saturating the concrete is forced inwards, creating a hydraulic bursting pressure.

A third, that the bursting is due to the formation of ice lenses in a manner similar to those leading to the "frost heaving" of ground.

For the engineer, the obvious course is to avoid the use of any detail which might lead to saturation of the concrete. Fine gradings which require a high proportion of water to ensure workability, and consequently absorb water readily, should be avoided. Projections on which water might collect, copings or cills, should have a considerable slope and not be made up with mortar subsequently.

At the other extreme, heat ultimately destroys all structural materials ; their relative merits depend on the amount of heat they will accept without serious loss of strength, that is on the period and intensity of the temperature. A fault, if such it can be called, can only be due to the selection of an unsuitable aggregate. There is a considerable difference in the expansion due to heat of cement and natural stone, and for fire-resisting purposes the more nearly these are akin in structure and composition, the better. One reason being that with reinforced concrete the longer the covering remains in position despite its own loss of strength, the longer it will be before the more vulnerable reinforcement reaches the 500° or 600°C. at which its strength is entirely lost.

The most obvious faults in external work generally take the form of spalling or splitting of the concrete due to the corrosion of the reinforcement, moisture having reached the steel either through porosity of the concrete, through faulty construction joints or by way of cracks primarily due to either stress or shrinkage.

Fig. 5.—Bridge parapet

The protection given to reinforcement by concrete is mainly due to its alkaline character. Porosity, whether due to bad mixing, segregation, incomplete consolidation, too little cement or other causes, allows leaching to occur and the protection afforded to be gradually removed.

And here it may be suggested that reinforcement is too often inserted where it is not really necessary. The excuse may be to provide some kind of anchorage to any new concrete which might be required, if when striking the formwork or through some other accident a projection was damaged. There may be places where a provision of this kind is desirable, but considering the reliance placed on the diagonal tension strength of concrete in slabs and some beams, it would seem that

Fig. 6.—Concrete washers

for minor projections, such as cills, reinforcement is not needed and does more harm than good.

Construction joints are not easily made water-tight. To consolidate the concrete properly it is usually necessary to use a stop board through which reinforcement will have to pass. Theoretically, this board should be in a plane normal to the principal compression stress, be fitted so as to prevent honeycombing due to leakage of cement, and be firmly held to withstand proper consolidation. On removal of the board, the surface may be too smooth to give a good key and, since cement bonds better to a clean aggregate than to old cement, the surface should be hacked, cleaned, and given a coat of neat grout before the new concrete is punned against it, the process being simplified if a vibrator is available. Even then the joint is not likely to be the strongest part of the concrete, and any big moisture or temperature changes may cause fracture, for which reason subsidiary additional reinforcement is sometimes inserted.

Cracks due to shrinkage or bending usually occur in planes normal to the main bars and in line with a stirrup or binding, the complicated bond stresses on the stirrups no doubt being largely responsible. Small movements of the bars, especially of stirrups, after the concrete has stiffened, can lead to the formation of cavities along the line of the reinforcement, and indicate the desirability of framing it up rigidly by spot welding or other means, and keeping it in position by concrete washers of some kind.

Moisture entering a crack at a stirrup can flow down until it reaches the main bar and then, since the bond on the main bars adjacent to the crack must of necessity have failed, there is the likelihood of the water finding its way along the bar. How far this is rendered more probable by the suggestion in some specifications that

where reinforcement is congested, a wetter concrete may be used, is open to question.

A point at which faults are frequently obvious is in external columns at floor level ; there are usually construction joints both above and below the beams, reinforcement is congested, the column bars are both cranked and spliced, and bond stresses are high. All these factors combine to lessen the rigidity of the junction and increase the likelihood of crack formation.

Fig. 7.—Progressive corrosion

Cracks may be due to moisture and temperature movements. If a foundation is virtually fixed and the floors expand or contract, considerable stresses are set up in any projecting balconies and in the end columns of long buildings, with results very similar to those referred to in the last paragraph ; the stiffer these supports are made, the greater will be the stresses induced in the floor. In a concrete wall the tendency will be to rock the piers between the windows longitudinally, which may be a contributory cause of the diagonal cracks which sometimes occur at the head and cill of window openings.

Fig. 8.—External columns

Whatever the cause of cracks may be, there is little doubt that for external concrete structures more cover than is specified in early rulings is desirable, if the reinforcement which is the source of strength is not to be the cause of failure through corrosion. Increased cover is advantageous in several ways ; there is more possibility of consolidating the concrete round the bars and therefore less chance of porosity or honeycombing ; there is greater resistance to moisture penetration and less chance that careless vibration or punning will bring the bars too near the surface.

This need for greater cover externally is recognised in the new Code by the virtual recommendation of a minimum cover to all reinforcement of 1 inch. In the United States the suggestion is 1½ in., with mention of the desirability that all reinforcement shall be adequately secured in position by concrete or metal chairs or spacers.

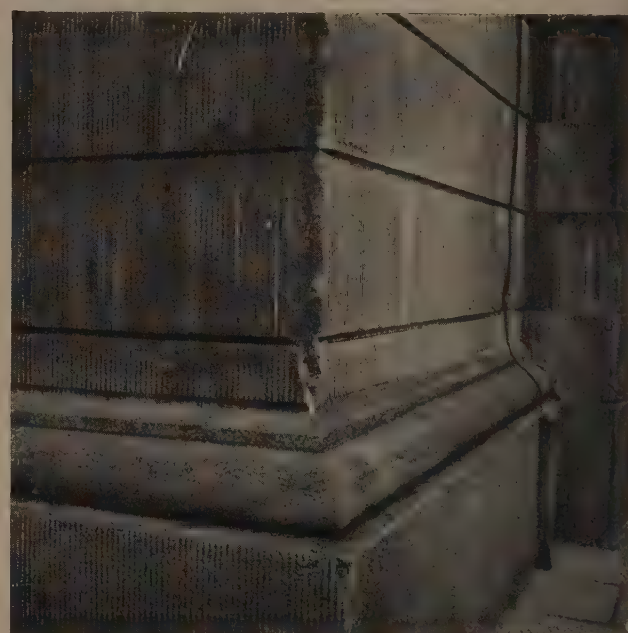

Fig. 9.—Insufficient cover

For water retaining structures the Institution of Civil Engineers Code recommends a minimum cover of 1 in. when using 1 : 1.6 : 3.2 concrete, and 1¼ in. for 1 : 2 : 4 concrete, accompanied by advice as to a reduction of stress in both steel and concrete. It is true that for this particular class of structure there is virtual certainty that the full design load will be applied continuously, but this is equally true of large span bridges where the proportion of dead load is high.

To summarise : there is no evidence to show that within a building reinforced concrete as designed in the past is not virtually permanent. Externally, where subject to the weather and a wider range of temperature, there are indications that special consideration is needed. In traditional building the nature of internal walls and finishings is very different to those outside, and it may be that there should be some revision of present practice as to the cover given to reinforcement and the stresses employed.

How far such provision is necessary must be left to the judgment of the engineer, who must rely largely on personal observation of faults in existing structures and identification of their probable cause.

Unusual Design for a Large Constructional Shop*

By F. R. Bullen, B.Sc., M.I.C.E., M.I.Struct.E. (Member of Council)

Introduction

The work which forms the subject of this short paper is the first stage in the construction of a new factory for Messrs. Ashmore, Benson, Pease & Co., at Stockton-on-Tees. The entire new works cover an area of approximately 100 acres, and are expected eventually to provide a covered floor space of about 40 acres. The area of the new constructional shop in this first stage is approximately 132,000 sq. ft.

The new works will have, as shown in Fig. 1, railway and road access ; the rail access connects to an extensive stockyard and also to the area where the finished articles are assembled. The works will be complete with washing and lavatory facilities, canteens, vehicle storage buildings, electrical sub-stations, boiler houses and various offices.

Site

Prior to the design of the foundation works an investigation was carried out, which showed that the quality of the clay near the surface was extremely variable, and worth no more than ¾ ton per square foot. A pre-piling survey was then put in hand, from which it was found that the site consisted almost entirely of clay overlain by a few variable deposits of sandy clay and stones. As a result of these investigations it was determined to support all heavy loads upon piles. The first stage of the constructional shop, which is to be described later, together with the foundations of the services building and office building, were generally carried on driven piles totalling 630 ; the majority of these piles were subjected to compressive loads of about 45 tons. In addition, 42 bored piles were installed, carrying about 30 tons each, in situations where the normal vibration would have damaged an existing sewer.

General Arrangement

A plan of the main constructional shop is shown in Fig. 2, from which it will be seen that it consists of four transverse bays, each 85 ft. 6 in. span, bounded at each end by a longitudinal 70 ft. span. In all six bays, overhead electric cranes of various capacities are provided, the height of the crane rails in the transverse bays being 30 feet, and in the longitudinal end bays provision has been made for two 50-ton cranes per bay. As a result of these fairly heavy crane requirements, combined with the considerable heights of crane rails and the somewhat unusual arrangement of one bay entering another bay at right-angles, a number of problems arose. For instance, the crane beams serving the end bays have to span 85 ft. 6 in. on to the isolated stanchions, which, in addition, were carrying the cranes from the transverse bays, which not only rested upon these columns but passed into the longitudinal bays. As a result, these columns carry maximum loads of 267 tons, bending moments amounting to 117 foot tons, and shear forces of 37 tons. It may also be mentioned that all columns have been designed to permit the attachment in the future of jib cranes giving rise to a bending moment of 60 foot tons.

Fig. 3 shows a typical cross-section of the four transverse bays, and Fig. 4 shows a typical cross-section of the two longitudinal bays with a longitudinal section of the transverse bays.

The desire of the company was that the building should be of the most modern type, and for this reason large clear spans uninterrupted by roof trusses or other unsightly members, together with a proper use of welding, seemed to follow. The original sketches were prepared on the basis of the frames being continuous at knee or gutter level, and probably fixed at the bases, although the possibility of hinged joints at these points was not overlooked. At that time it was also thought that the slopes of the roofs would be kept to about 10° with the horizontal, as this has been found to give a very economical arrangement, and in any case reduces the volume of the building to a minimum. At this stage also considerable thought was given to the glazing arrangements ; a low roof slope has many advantages in relation to the natural lighting of a building, since it makes available the largest possible area of the illuminating hemisphere. As the scheme developed, however, it seemed desirable to increase the roof slopes to 15°, largely on account of the better æsthetic shapes so produced ; the flatter slopes seemed to be too unshapely for the large spans and heights required by the general outline. Accordingly, 15° was adopted and light curves prepared with various arrangements of roof glazing. The original intention had been to use what has been described as the "pepper-pot" form of roof lighting, which has many advantages in lower roofs, and leads to a very even distribution of lighting at the working plane. It was found, however, in the present case that the great heights of the roofs made the "pepper-pot" formation unnecessary, and continuous runs of glazing gave satisfactory lighting curves. In consequence, the continuous glazing runs were adopted, and it is suggested that the results shown in Fig. 5 would seem to justify this arrangement.

The roof slopes were covered with V-beam R.P.M. sheeting, the V-beam form being adopted so as to allow the purlin spacings to be as large as possible. Whilst the general features of the building are being described attention should be drawn to the large area of glazing at the side of the north longitudinal bay, which was a feature desired by the company and heartily endorsed by the architect, Mr. G. A. Jellicoe, F.R.I.B.A., whose advice had been sought by the company in connection with matters of lay-out and the external appearance of the buildings, together with certain detailed features of the offices and canteen.

The structural features of the building fall naturally into two parts, (1) the transverse bays, and (2) the longitudinal bays, and these will be separately described.

(1) Transverse Bays

As mentioned above, the original intention had been to design the buildings as a series of rigid frames. Detailed consideration of the problems involved, how-

Paper to be read before the Institution of Structural Engineers, at 11, Upper Belgrave Street, London, S.W.1, on Thursday, March 27th, 1952, at 6 p.m.

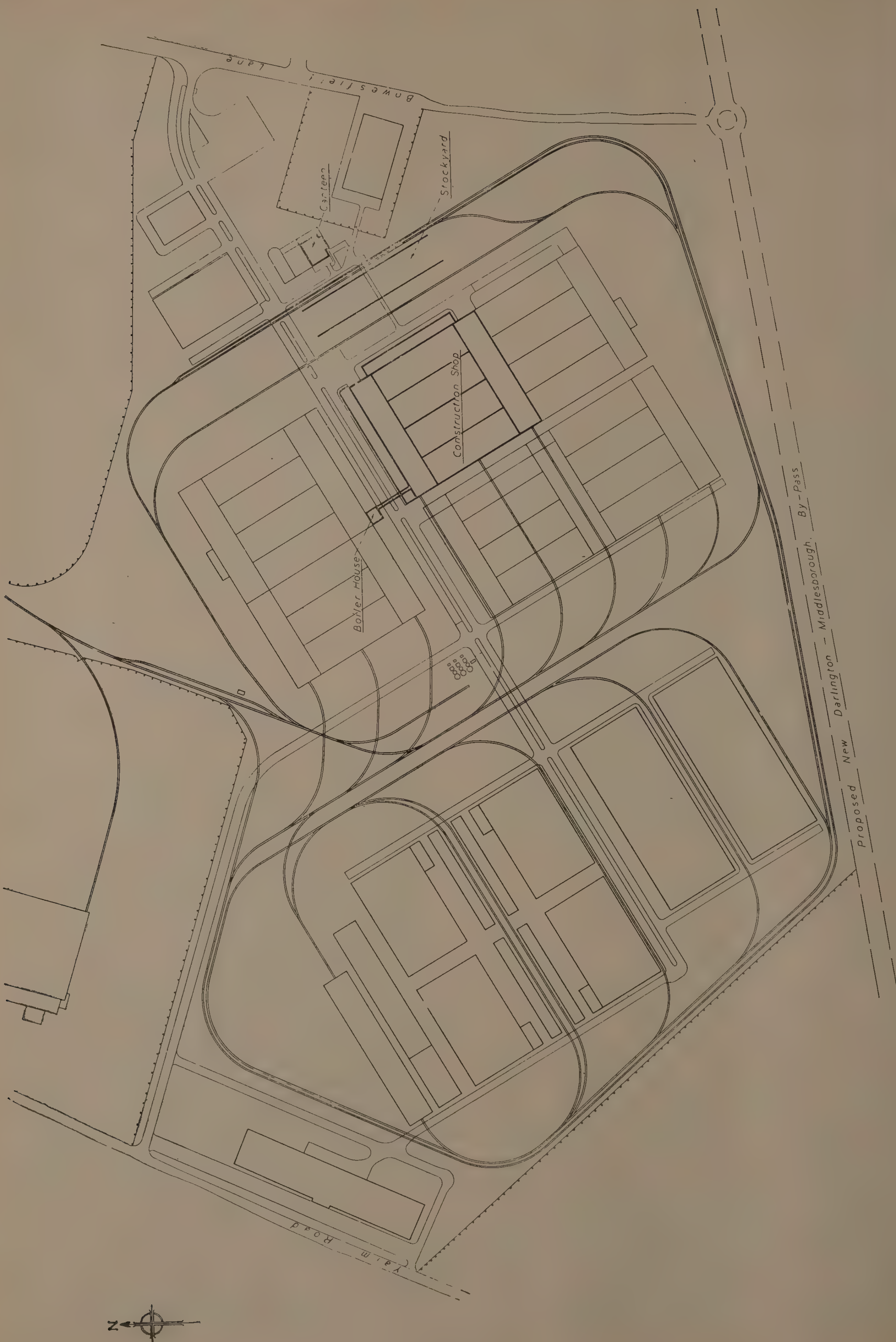
Bowesfield Lane
Canteen
Stockyard
Construction Shop
Boiler House
Proposed New Darlington – Middlesborough By-Pass
Yarm Road
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Fig. 1

ever, soon drew attention to the fact that at a height of 30 ft. and with cranes such as those envisaged, there was a fundamental error in the provision of a structure arranged in such a way that the forces imposed upon it caused all the members to be loaded both directly and

roof rafter was primarily provided for the purpose of protecting the building from the weather, it seemed unnecessary to have to increase its size to take up the crane forces, and considerations of this type led to the conclusion that it was more logical to transfer the

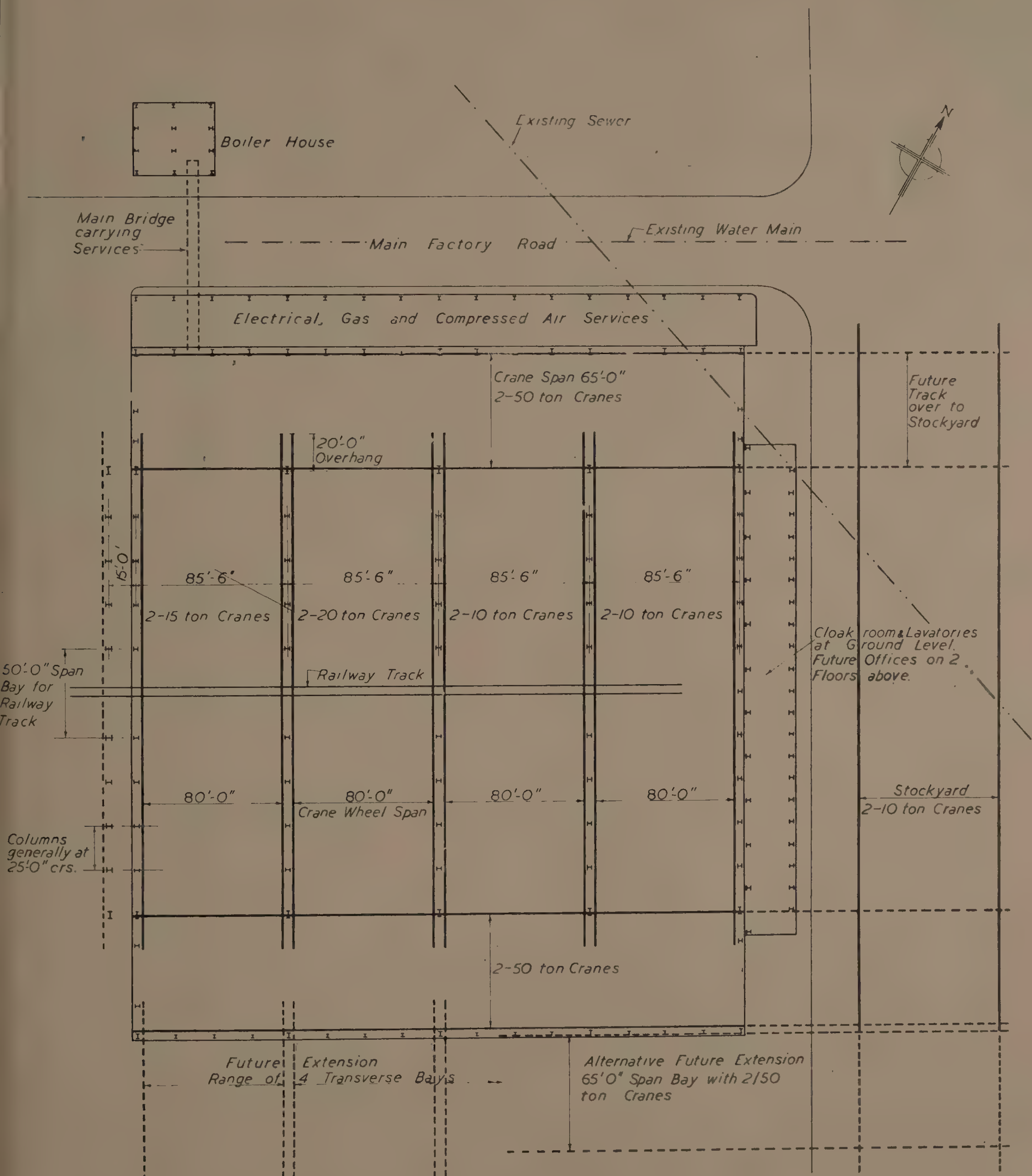


Fig. 2

indirectly. For instance, had rigid frames been employed, a crane reaction occurring at a column would have caused not only that column to be loaded in compression and bending but also the roof rafter, and, of course, the knee-joint between the two. Since the

crane forces direct to the ground through the columns. Accordingly, the idea of tapered columns developed, designed substantially as vertical cantilevers and capable of absorbing the vertical crane loads and the horizontal surges as well as the wind forces. Fig. 6 is a diagram-

matic cross-section of the four bays, in which are shown the forces which the various columns have been designed to resist, and the articulation of the members. It will be noticed that the horizontal crane and wind forces have been concentrated on particular columns, and this has been effected by the use of "sloppy" fits on the remote crane wheels. It will also be noticed from this that, except for the outside columns, the wind and snow loads are the only forces acting upon the roof members, which, in consequence, can be correctly described as constituting a large umbrella over the working space below. The outside columns, being hinged at both the top and bottom, receive support against the eccentric crane reactions from the roof rafters. A further development of this structural outline then took place. In order to economise in the sizes of the roof rafters, a degree of continuity was examined and it was found possible to treat the rafters of the outer bays as continuous over the first lines of columns, thus giving rise to a form of double cantilever construction. This, in turn, has led to the use of standard rolled sections in many of the members and a reduction to a minimum in the use of built-up welded sections. Figs. 7 and 8 show this form of construction, and it is suggested that a very light result has been achieved, giving rise to a pleasing appearance and complete freedom from all the usual roof truss decorations. Furthermore, the use of larger members instead of the multiplicity of small angles has reduced the maintenance painting costs to a minimum. The purlins span 25 feet between adjacent frames.

An approximate calculation was made to discover what would have been the weight of steel involved, had the four transverse bays been constructed as four continuous rigid frames. Whilst it is always a little difficult to compare approximate designs with final designs, the figures have shown that the design adopted is about 10 per cent. lighter in weight of steel than would have been the case if the rigid frames had been used. The principal economy, however, consists in the use of simple rolled sections to a very much greater extent than would have been possible for the rigid frame, with the result that the average price per ton of steel has been quite low.

(2) Longitudinal Bays

The design of the longitudinal bays was, of necessity, quite different from that of the transverse bays. In order to give a similar appearance and in order to economise as much as possible, it was desired to use rolled sections, and accordingly, it became necessary to consider the rafters as supported at the ridge and the eaves. On the external wall of the north bay the eaves support took the form of stanchions, but the internal supports consisted of latticed girders spanning 85 ft. 6 in. between the large main stanchions. The spreading effects of the inclined rafters were resisted by members in the plane of each rafter slope which formed large girders in the planes of the roof slopes. The reactions from these girders were transferred to each main stanchion by means of the deep stiff welded ribs, shown in Fig. 9, and these ribs were also considered to transfer the wind forces from the external walls across the bay to the main stanchions. By these means the horizontal forces developed by the cranes in these bays were shared between the two crane beams, but were wholly transferred to the main stanchions, together with the horizontal wind forces. Fig. 10 is an isometric view of the intersection of the transverse and longitudinal bays, and shows up the lattice girder arrangement and a number of other features. The external columns were then

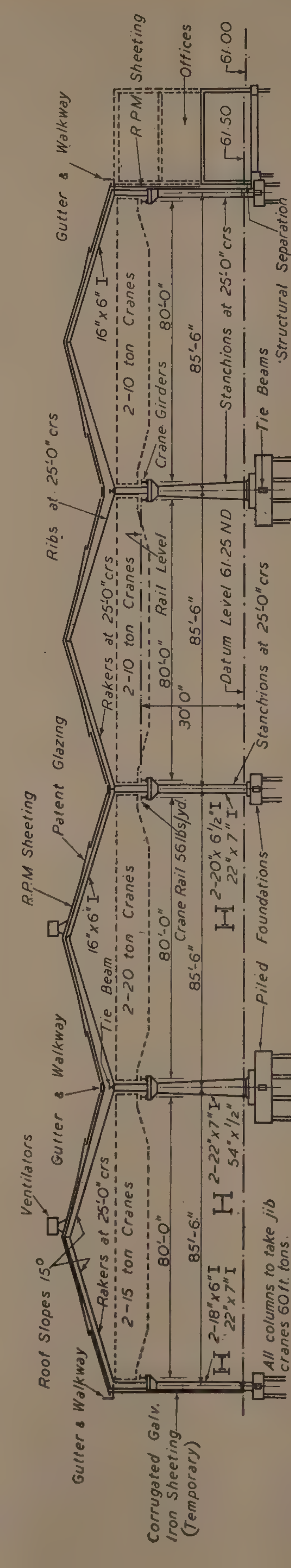


Fig. 3.—Typical Cross Section

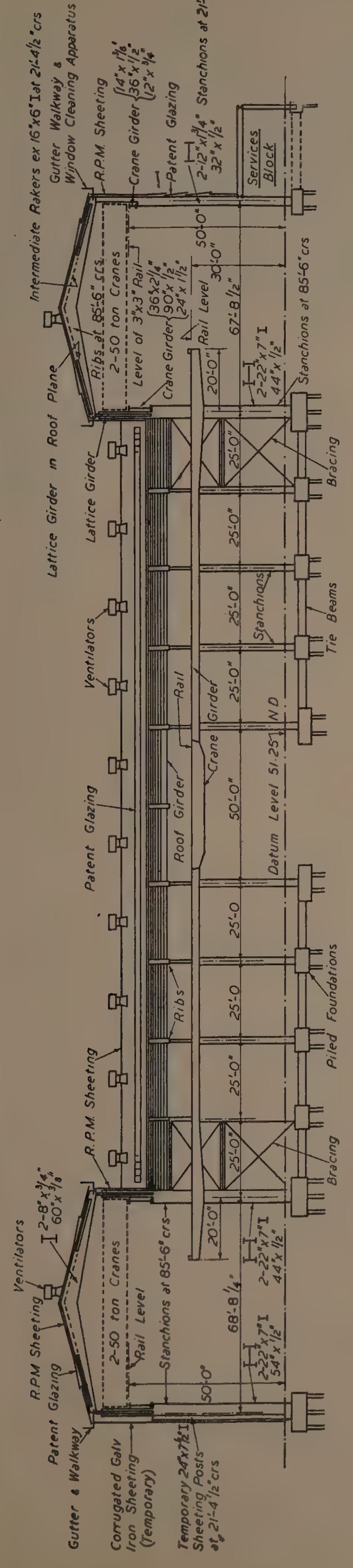


Fig. 4.—Typical Longitudinal Section

treated as simply supported from ground to eaves level for wind forces, although they all had to take their full vertical crane reaction.

On the south side, however, the longitudinal bay, whilst it has been temporarily sheeted at the present time, has been designed to provide for either a similar longitudinal bay or a series of transverse bays in the future. In consequence, the main stanchions were provided at 85 ft. 6 in. centres, supporting the outer crane beams and the main roof ribs ; the intermediate roof rakers were supported upon deep latticed girders spanning between the main stanchions and arranged also to give support to the sheeting rails carrying the temporary galvanised corrugated sheeting. In addition, removable stanchions were provided spanning from ground up to the underside of the latticed girder, which not only gave support to the sheeting rails but, in addition, were fitted to horizontal wind girders, which in turn spanned between the main stanchions. In order to absorb the longitudinal forces set up by the cranes in this bay, heavy diagonal bracings were introduced on the end bays which brought the main latticed girder into use as a portal frame.

Reference may be made at this stage to the large crane girders in the longitudinal bays. Each of these girders was 85 ft. 6 in. long and weighed about 22 tons ; it consisted of a 90 in. $\times$ $\frac{1}{2}$ in. web, with a top flange 36 in. wide by $2\frac{1}{4}$ in. thick, and the bottom flange 24 in. $\times$ $1\frac{1}{2}$ in. thick. The flanges were tapered to 18 in. width adjacent to the supports. The flanges and web were welded directly to each other, using $\frac{1}{2}$ in. fillets for the compression flange and $\frac{3}{8}$ in. fillets for the tension flange. $\frac{1}{2}$ in. flat stiffeners were provided throughout the length of the girders at approximately 4 ft. 9 in. centres, and near the centres of the spans intermediate stiffeners $\frac{1}{2}$ in. thick were introduced to give additional support to the compression flange against buckling due to lateral crane forces. The girders not only rested upon the stanchion cap plates but were braced to the upper limbs of the stanchions by means of stiff diaphragm connections, the object being to ensure that all lateral forces arising from the crane movements would be effectively transferred to the stanchions. The crane rails consisted of 3 in. square steel billets, tack welded at 18 in. centres to the top flanges of the girders. The result of these arrangements is shown in Fig. 11, which is a photograph of the northward longitudinal bay. It is suggested that a very clean open form has been achieved which is simple and economical to paint initially and also to maintain. It is difficult to comment upon the economies resulting from this arrangement. The absorption of the horizontal forces gave rise to some considerable thought, and the arrangement adopted seems to be the most economical having regard to the general arrangement of the structure. Presumably, if the more orthodox arrangement had been used this might have resulted in a series of roof trusses instead of the rakers and arched ribs, each truss supported in the same way as the corresponding roof member in the design which is being described. This would have entailed considerable stiffening of the trusses at the main stanchions, unless the lateral forces which are now being transferred across the bay had been transferred by means of a substantial horizontal girder in the plane of the ties of the trusses. The use of roof trusses, however, would have involved greater painting and, therefore, maintenance, and in any case does not give the clean open lines of the present construction. It is worth pointing out also that it has been possible to use, in combination with welding, simple rolled sections almost entirely, and

in consequence, the average price per ton of steelwork has been very economical.

Stockyard

The design of the stockyard steelwork has followed what might be described as orthodox lines. The crane spans were made 80 ft. so that the cranes could be interchangeable with those in the transverse bays of the shop, and provision has been made for doubling the stockyard in the future. In general, the stockyard supports are at 50 ft. centres, the transverse crane surge forces being absorbed by horizontal surge girders attached to the main running girders. These crane girders were all welded, as were the trestles. An interesting provision has also been made for the future; where the stockyard crane gantry lies in front of the two longitudinal bays of the main shop, the trestles have been designed in such a way that they can be extended upwards to form the supports for future extensions of the large crane girders in those bays. This provision would enable the 50-ton cranes in the longitudinal bays to pass out of the building and over the stockyard, should the need arise in the future for much heavier loads to be transferred into the building. These provisions had their effect upon the stockyard gantry design as also did the foundation requirements. In this latter respect, owing to the necessary use of piles for foundations, it was desirable to load the piles fully, and in consequence, the stockyard gantry supports were spaced at 50 ft. centres. Fig. 12 shows a view of the stockyard as it is now.

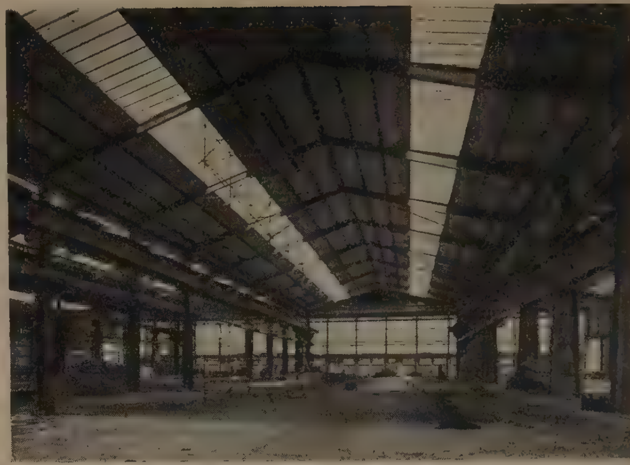


Fig. 5

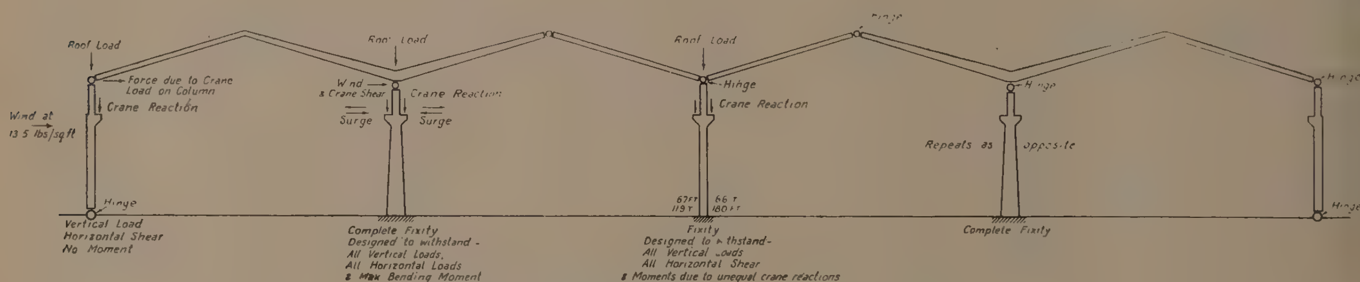


Fig. 6.—Transverse bays—Articulation diagram

Boiler House and Pipe Bridge

It will be seen on reference to Fig. 2 that on the north side of the main factory road the first section of the boiler house has been constructed. This boiler house is a steel-framed structure supported upon a reinforced concrete raft, and is connected to the constructional shop by means of a pipe bridge which spans on to the boiler house and also on to the constructional shop and services building, the total distance being 101 ft.

There is nothing particularly novel about the boiler house design other than that it has a nominally flat roof, and incorporates steel floors at elevated levels to give access to the various mechanical features of the boiler plant. The pipe bridge also is fairly straightforward in design; the sides are built up of angles to form latticed girders, to which additional angles and joists are attached to support the timber walkway which gives access to the many service supply pipes which cross the road through this bridge. The whole of the bridge is covered with Robertsons' Protected Metal sheeting. Fig. 13 shows a photograph of this bridge and the boiler house structure.

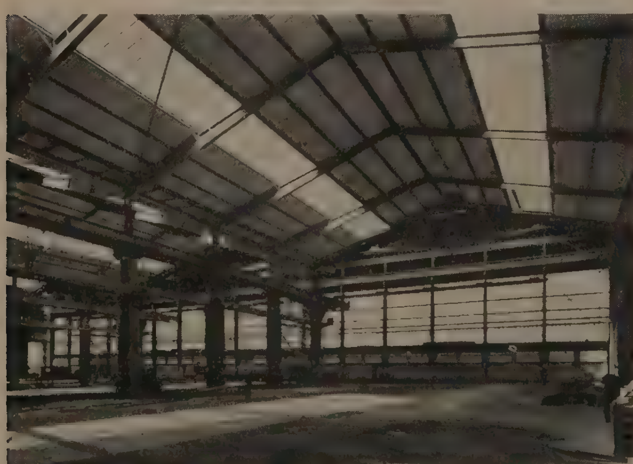


Fig. 7

Fig. 14 shows a photograph of the steelwork of the first section of the canteen building. This is a small building measuring 70 ft. × 54 ft. In the back section is the kitchen and in the front section the space for service of meals. The steelwork arrangement was dictated by the architectural requirements, which envisaged an entirely glazed frontage free from obstruction of steelwork.

One detail of importance was carefully ensured, namely, that the trusses in the kitchen section consisted of single angles to facilitate painting and avoid lodgments for condensation moisture. This is an extremely important feature in buildings such as kitchens, where corrosion rapidly sets in if steelwork cannot be readily maintained.

The steelwork of the office block and services building was entirely straightforward and consisted of simple beams and stanchions. The office block was supported on piles with reinforced concrete foundations, and

ovision was made for the addition of two further oors in the future.

Erection Problems

The first reaction of the contractors upon receipt of e drawings was summarised by them as follows :

"Preliminary perusal of the drawings indicated a ructure of most unusual design, and it was evident that ecial considerations would have to be given to the ection of it."

The erection problems, therefore, were considered ider the following headings :—

(a) Special features of the structure.
(b) Stability of the structure during erection.
(c) Size and weights of bars most convenient for ection and welding on site.
(d) Access facilities for materials.
(e) Erection procedure.

(a) Special Features

OUR TRANSVERSE BAYS

(1) Roof construction in pairs of balanced ribs.
(2) Balanced ribs consisted of one fixed apex and one iding apex.
(3) The balanced rib was pin jointed at all three ipports.
(4) The central stanchion to the balanced rib was ced at the base to resist overturning. The other anchion to the balanced rib was hinged at the base well as the cap.

ORTH AND SOUTH LONGITUDINAL BAYS

(1) Roof to each high bay designed in four longitudinal ctions.
(2) Each longitudinal section comprised in effect two ate ribs, one at each end, and three intermediate ist ribs.

Fig. 8

(3) Intermediate ribs braced by a system of rafter bracing which was only complete when the end plate ribs were fixed.

(4) Main columns on the side of longitudinal bay adjacent to the transverse bays were braced vertically to the first column of the transverse bays in each case. This bracing was intended to take the wind load from the roof through the main plate roof ribs. The north set of bracing was incomplete without the south set of bracings. The north set was designed for wind from the North, and the south set for wind from the South.

(b) Stability of Structure during Erection

(1) FOUNDATIONS

The special features described above made it imperative that all foundation bolts should be grouted

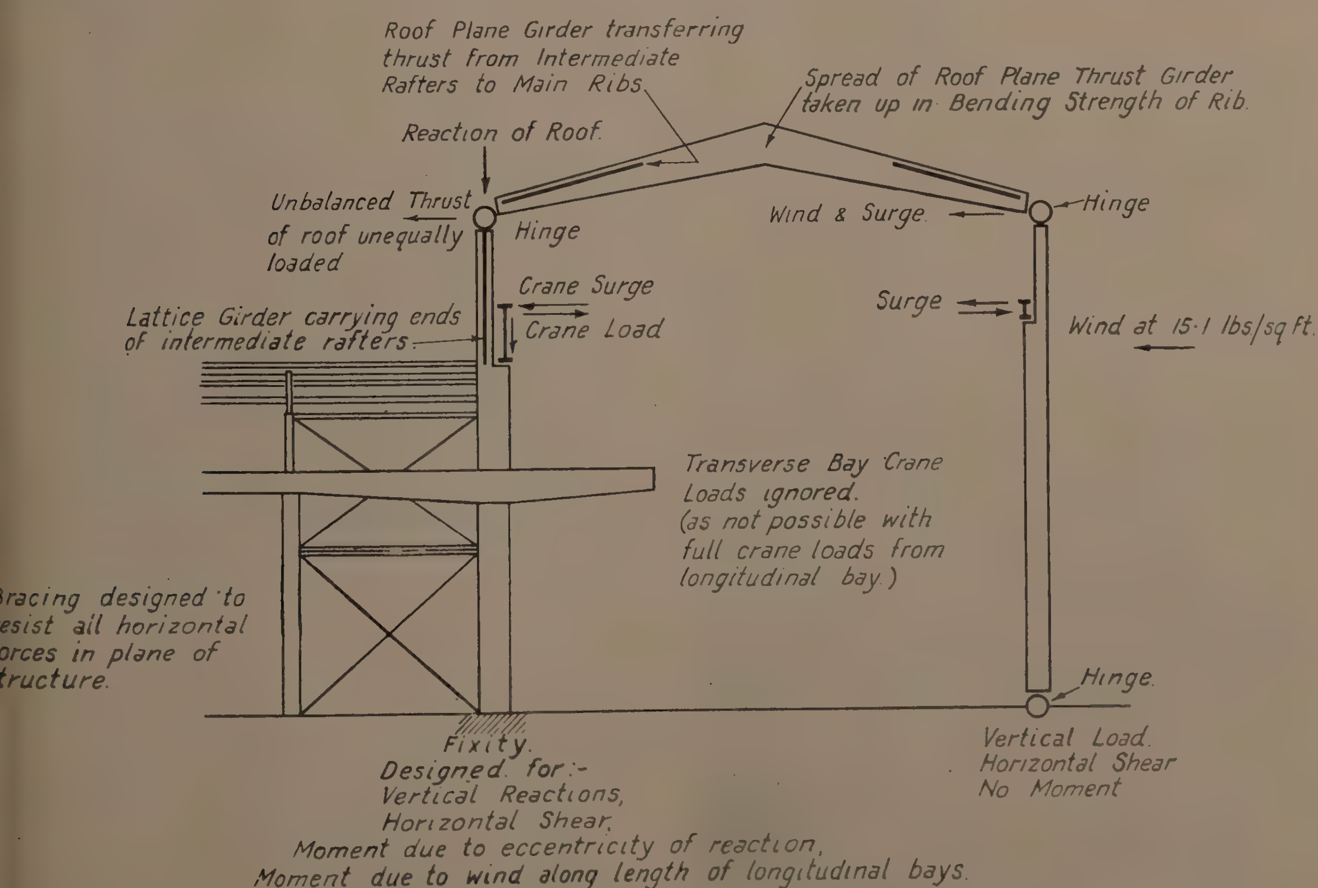


Fig. 9.—Longitudinal bays—Articulation diagram

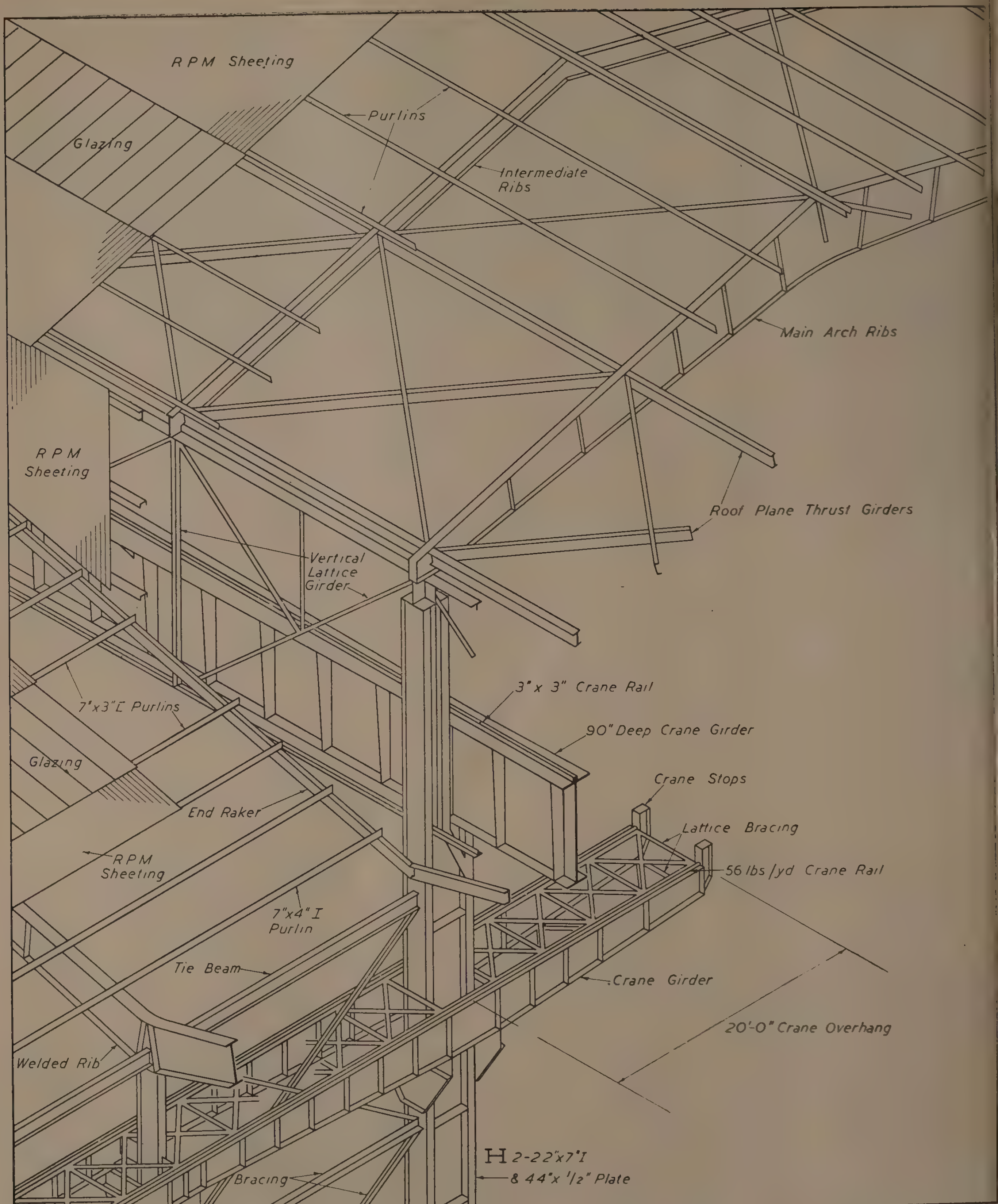


Fig. 10

solid with the foundation block before any stanchion was erected. It was decided, therefore, to make steel templates of the various stanchion bases. These templates not only held the bolts in the correct location horizontally, but held the bolts at the correct height. The templates in these respects were an exact replica of the stanchion bases. The templates were set for location and level, and the bolts were concreted up to the level of the concrete foundation. The templates were then removed and the stanchion fixed on level steel packings. By this means the stanchion base was rigidly fixed in location. Plumbing was accomplished by varying the relative levels of the packings.

(2) Transverse Bays

The roof ribs over two bays had to be erected as a unit if the balanced design was to function. The ribs over two bays were assembled and welded on the

ound, to the correct span and height dimensions. A refully designed and applied 8 point suspension lift en raised them into position.

) Longitudinal Bays

(a) The fact that the rafter bracing was incomplete itil the end plate ribs and three intermediate ribs were

Fig. 11

xed and braced made it necessary to restrain the itermediate ribs against opening out and increasing ie span dimension at the shoes. The desired restraint as ultimately provided by a horizontal tie channel irried on suspenders from the joist rafters and attaching iereto just above the shoes.

(b) The fact that the vertical bracing against wind at ie main columns was designed to function in one irection only, and was not able to be fixed until part the transverse bay was erected, made it necessary to

Fig 12

strain the building with guys in a north-south direction each main column as erection proceeded.

(c) Size and Weight of Bars

(1) The only bars to present any problem in this spect were the heavy crane girders and plate ribs in the ngitudinal bays and the balanced ribs in the transverse ys.

(2) Investigations proved that the main crane girders uld be handled at site in one piece and that transport in one piece was possible to the site. The girders were therefore completely fabricated in the works.

(3) Plate ribs to the longitudinal bay were broken down to three pieces and assembled and welded on site to comply with transport regulations.

(4) The balanced ribs in the transverse bays were broken down into eight parts for each pair of bays, and were welded and assembled on site to comply with transport regulations.

(d) Access Facilities for Materials

Investigation and discussion on site enabled arrangements to be made for road access on the whole of the north, west and east sides.

Fig. 13

(e) Erection Procedure

Longitudinal Bays

One 15-ton electric and one 5-ton electric crane, each mobilised on bogies and with 100 ft. jibs, were used for this section. The cranes traversed in the longitudinal direction of the bay. The width of the longitudinal bays was insufficient to accommodate the cranes side by side, and accordingly they were erected at suitable

Fig. 14

distances one behind the other in the longitudinal length.

Transverse Bays

Two 5-ton electric cranes, each mobilised with 100 ft. jibs, were used for this section. The cranes were erected

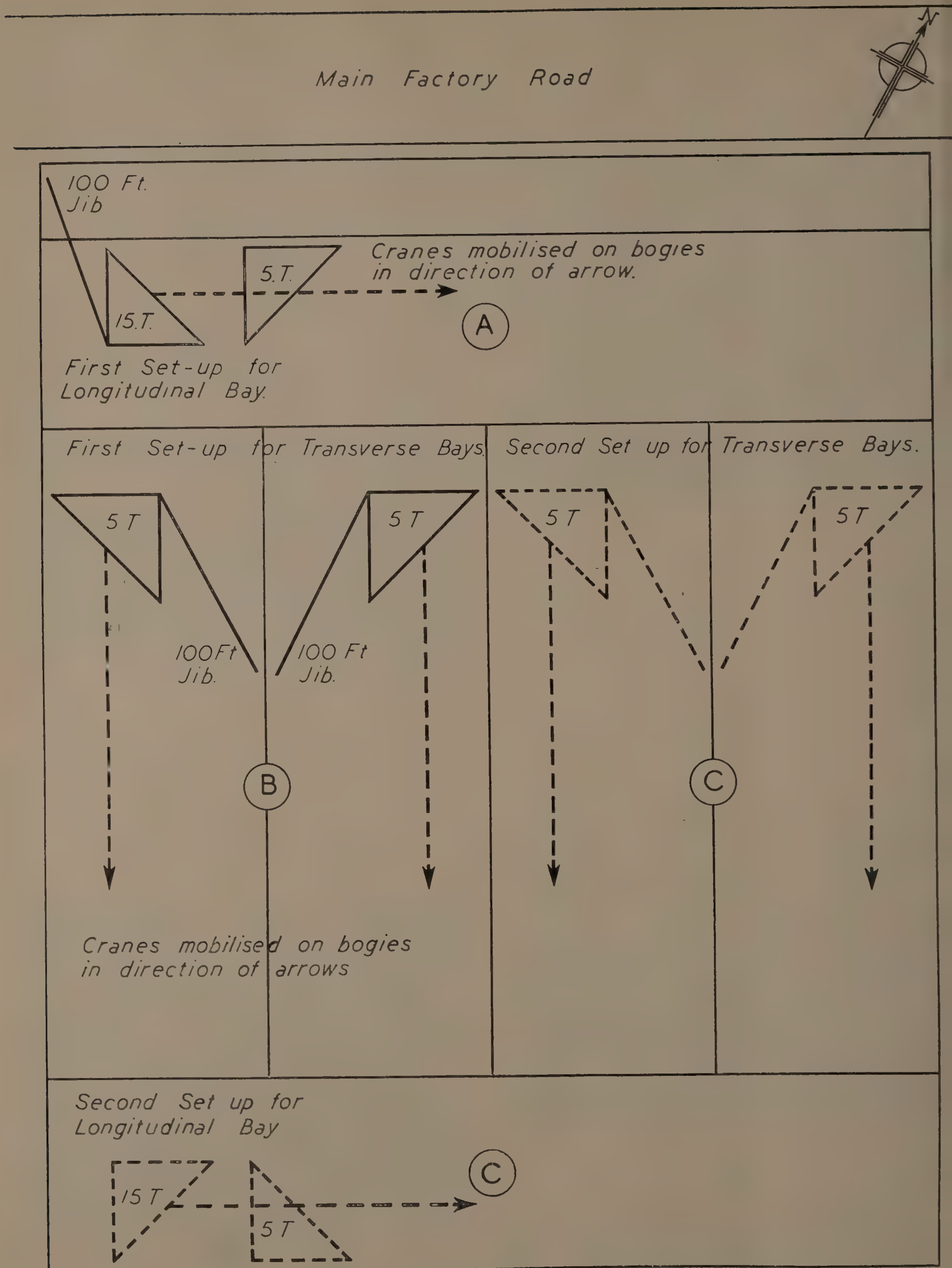


Fig. 15 (a)

side by side on a transverse line, one in each bay of one pair of balanced bays. The cranes were moved down the longitudinal length of the building as erection proceeded. An additional 5-ton diesel crane working on caterpillar tracks was later introduced for gener[al] purposes.

Erection commenced at the west end of the nor[th] longitudinal bay and proceeded in an easterly directio[n]

taking in the services building and the first bay of the transverse bays ; the latter was essential for stability considerations. On completion of erection of the north longitudinal bay, erection commenced at the north end of the west pair of transverse bays. During the period the west pair of transverse bays were being erected, the two erection cranes in the north bay were dismantled, and re-erected at the west end of the south longitudinal bay.

When the erection of the west pair of transverse bays reached the south longitudinal bay, the two 5-ton cranes which had erected the west pair of transverse bays were dismantled by the erection cranes set up in the south longitudinal bay. These two 5-ton cranes were then re-erected at the north end of the east pair of transverse bays while the erection of the south longitudinal bay proceeded. It was estimated that half the longitudinal length of the south longitudinal bay would be erected by the time the erection of the east pair of transverse bays reached the south bays. This estimation proved correct, and the two 5-ton cranes erecting the east pair of transverse bays were dismantled by the south longitudinal bay erection cranes and sent off the site.

The erection of the second half of the south longitudinal bay was then completed.

The average rate of erection of steelwork was 67 tons per week ; this was less than the original estimate on account of these specific factors :

(1) Bad weather.

(2) Irregular delivery of steel due to shortages of specific sections.

(3) The first arrangements made were barely suitable for the job, particularly in the north longitudinal bay ; the south longitudinal bay, however, was erected much more expeditiously with the same plant after the difficulties experienced in the north bay had been overcome.

Fig. 15 shows, diagrammatically, the erection arrangements.

The foregoing description of both the design and erection of this somewhat unusual structure will have exposed a number of problems. The most important difficulty which arose was that relating to the erection of the north and south longitudinal bays. These difficulties related to the large lattice girders in the rafter plane, and they arose because, until those girders were fully bolted up and completed, the intermediate joist section rafters were free to spread under their own weights. As has been mentioned above, this tendency was prevented by the introduction of temporary ties near the eaves level, but the first few rafters erected were not temporarily supported in this way, and consequently gave rise to some quite difficult problems due to the spread. It may also be mentioned that the same problem was aggravated by the fact that difficulty was experienced in fabricating the stanchions in perfectly straight lengths ; there was a tendency for the smaller limbs at the tops of the stanchions to deflect outwards on account of the welding, which not only assisted the rakers in spreading but in consequence led to a somewhat uneven ridge line. This difficulty arose particularly on the north longitudinal bay, where the feet of the rafters are supported upon the smaller and more slender stanchions. The opinion was expressed by the contractors that the intermediate rafters should have been designed against spreading during erection, but the author does not agree with this, although he agrees that the provision of temporary ties, as were introduced subsequently during erection, was essential.

Another important point which arose related to the stability of the longitudinal bays during erection, and prior to the erection of the transverse bays. As has been remarked, and as may be seen from Fig. 9, the lateral stability of the longitudinal bays depends upon the bracing in the first bay of the transverse bays ; furthermore, the stability of the north bay for wind blowing from the North is absorbed by this lateral bracing, but with the wind blowing from the South that bracing is no longer effective. In consequence of this, it became necessary to introduce cable anchorages temporarily to ensure stability during erection.

The author, in common with the contractors, feels that the bracing in the first bay of the transverse bays should have been capable of withstanding wind pressure in either direction. Notwithstanding this defect, however, which after all was quite readily overcome and really constituted part of the erection procedure, the design has produced a structure which it is hoped may form a worthy addition to British engineering.

Grateful acknowledgement is made to Messrs. Ashmore, Benson, Pease & Co., for their permission to publish the information contained in this paper, and especially to their works manager, Mr. T. K. Hargreaves, M.A., A.M.I.Mech.E., for his very real co-operation at all stages.

The contractors who supplied and erected the structural steelwork were The Cargo Fleet Iron Company, Ltd., and the author's firm, F. R. Bullen & Partners, in their capacity as Consulting Engineers to Messrs. Ashmore, Benson, Pease & Co., designed the structure of the building.

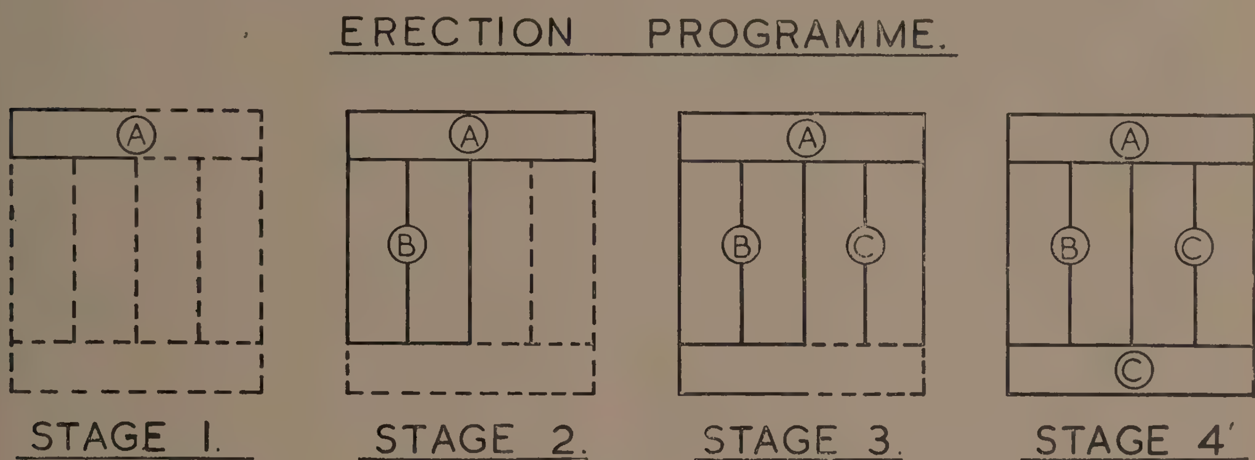


Fig. 15 (b)

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, January 24th, 1952, at 5.55 p.m., Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Students

CORBETT, Joseph Frank, of Bury, Lancs.
CUSSONS, Stanley Harold, of Stockton-on-Tees.
DURLEY, Anthony William, of Loughton, Essex.
GERRIE, Michael Bain, of London.
GILDER, Peter James, of London.
HALL, Raymond Sidney, of Radford, Notts.
HARDCASTLE, Frank, of Salford, Lancs.
HOOKHAM, Raymond Albert, of Eastbourne, Sussex.
LEWIS, Brian David, of Romford, Essex.
MASTERS, Patrick Anthony, of Derby.
METCALF, Leslie, of Preston, Lancs.
ROBERTS, Geoffrey Russell, of Chepstow, Mon.
YOUNG, John Frederick, of Romford, Essex.

Graduates

ABRAHAM, Ebenezer Adeniyi, of Bolton, Lancs.
BROOKES, Roy, of Chesterfield, Derbys.
BRYANT, Paul Alastair Verdier, B.Sc.(Eng.), Rand, of Pretoria, S. Africa.
COBB, Walter Clifford, of Middlesbrough, Yorks.
CROWDEN, Brian Bertram, of Birmingham.
DAVIS, Elsbury John, B.Sc.(Eng.), Natal, of Durban, South Africa.
DUKAS, John Desmond, B.Sc.(Eng.), Cape Town, of Cape Town, South Africa.
FLETCHER, John Richard, B.Sc. Wales, of Walsall, Staffs.
GREENHALGH, Fred, of Bolton, Lancs.
GROYER, Edward, B.Sc.(Eng.), Cape Town, of Johannesburg, South Africa.
KATHIRGAMAN, Kanagarathathinam, B.Sc.(Eng.), London, of Point Pedro, Ceylon.
KHARKAR, Meghashyam Shantaram, A.R.I.B.A., of Heston, Middx.
KING, Kenneth Herbert, B.Sc.(Eng.), London, of Wallington, Surrey.
MATHIAS, Francis Cecil, B.E.(Civil), Madras, of Madras, India.
NICHOLSON, Thomas Hedley, B.Sc.(Civil), Durham, of Newcastle-upon-Tyne.
PATTEN, John, of Wolverhampton, Staffs.
ROTHWELL, Leslie Herbert, of Eccles, Lancs.
SMEDLEY, Granville Barrie, B.Sc.(Tech.), Manchester, of Royton, Lancs.
SMITH, John Allan, of Bolsover, Derbys.
SZALWINSKI, Stefan, of London.
TAIT, Alexander, of Motherwell, Scotland.
THORBY, Neil Henry, of Hillingdon, Middlesex.
WARNER, John Spearman, B.Sc.(Eng.), Natal, of Johannesburg, South Africa.

Transfers

Students to Graduates

LIVERMORE, Gordon, of Cardiff.
McCAULEY, John Brian, of Newton-le-Willows, Lancs.
SUMNER, Douglas Barrie, of Wellington, New Zealand.

Graduates to Associate-Members

AKERKAR, Anand Ganesh, B.E.(Civil), Bombay, of Bombay, India.
LIEBENBERG, Algernon Charles, B.Sc.(Eng.), Cape Town, of Rosebank, South Africa.
MADAN, Madhukar Yeshwant, B.E.(Civil), Bombay, of Bombay, India.
MOORE, Stanley George, of Birmingham.
MOTT, John Charles Spencer, of Thornton Heath, Surrey.
OAKEY, James Richard, of Hamilton, Ontario, Canada.
SIMON, John, of Salford, Lancs.

Associate-Member to Member

SEIN MAUNG KYAW, M.Sc.(Eng.), Manchester, A.M.I.C.E., A.M.I.Mun.E., of Rangoon, Burma.

Member to Retired Member

MITCHELL, Arthur James, O.B.E., M.I.C.E., of Beaconsfield, Bucks.

OBITUARY

The Council regret to announce the death of John Nicholas Patrick CONLAN (Associate-Member).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of Harry Ellis WADLAND (Member) ; Daniel Maclelland LAIRD (Associate) ; Peter Henry Clifford WATSON (Graduate) ; Edgar Ronald MORGAN (Student).

EXAMINATIONS—JULY, 1952

The examinations of the Institution will next be held at centres in the United Kingdom and overseas on July 15th and 16th, 1952 (Graduateship), and July 17th and 18th (Associate-Membership).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1 :—

Thursday, March 13th, 1952

Ordinary Meeting at 6 p.m., when Mr. P. G. Bowie, A.M.I.C.E. (Member), will give a paper on "Faults in Concrete Structures."

Wednesday, March 19th, 1952

Joint Meeting with the Reinforced Concrete Association at 6 p.m., when Mr. F. S. Snow, M.I.C.E., M.I.Mech.E. (Past President), will give a paper on "Recent Industrial Development at Port Sunlight and Bromborough."

Thursday, March 27th, 1952

Ordinary General Meeting at 5.55 p.m. This meeting, which is for the election of members and is open only to corporate members of the Institution, will be followed by an Ordinary Meeting at 6 p.m., when Mr. F. R. Bullen, B.Sc., M.I.C.E. (Member of Council), will give a paper entitled "Unusual Design for a large Constructional Shop."

Thursday, April 24th, 1952

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m.,

when Mr. S. Mackey, M.E., Ph.D., A.M.I.C.E.I. (Associate-Member) and Mr. D. M. Brotton, B.Sc., Ph.D. (Graduate) will give a paper entitled "An Investigation of the Behaviour of a Riveted Plate Girder under Load."

Thursday, May 22nd, 1952

Annual General Meeting.

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EARTH RETAINING STRUCTURES CODE

"Civil Engineering Code of Practice No. 2—Earth Retaining Structures," issued by the Institution of Structural Engineers on behalf of the Civil Engineering Joint Codes Committee is now available and may be obtained from the Institution, price 15s., post free.

INSTITUTION AWARDS

The following awards have been made for papers read before the Institution and at the Branches during the Session 1950-51 :—

London (Headquarters) Prize

Lt.-Colonel G. W. Kirkland and Mr. A. Goldstein, for a paper on "Design and Construction of a Large Span Prestressed Concrete Shell Roof."

Midland Counties Branch Prize

Mr. H. E. Brooke-Bradley, for a paper on "The Problem of the Supporting Power of the Subsoil."

Northern Counties Branch Prize

Mr. N. J. Ruffle, for a paper on "Concrete Control of Small Works."

Scottish Branch Prize

Mr. H. B. Sutherland, for a paper on "Some Problems in Soil Mechanics."

Western Counties Branch Prize

Mr. N. G. T. Ball, for a paper on "The Reconstruction of the Colston Hall, Bristol, 1951."

Wales and Monmouthshire Branch Prize

Mr. G. R. Brueton, for a paper "The Fatigue of Materials Related to Design."

Yorkshire Branch Prizes

Professor R. H. Evans, for a lecture on "The Institution of Structural Engineers' Report on Prestressed Concrete."

Mr. W. Hunter Rose, for a lecture on "The Durability of Concrete."

Mr. R. Oates, for a paper on "The Structural Design of the Mediæval Cathedral."

Union of South Africa Branch Prize

Mr. C. Rigby, for a paper on "Moment Distribution."

NEW YEAR'S HONOURS

In offering their sincere congratulations to the following member on the distinction recently conferred upon him, the Council feel that they are also expressing the good wishes of the Institution.

Order of the British Empire—O.B.E.
Mr William J. Fitt (Member)

SESSIONAL PROGRAMME 1952-1953

The Literature Committee are now considering and selecting papers for inclusion in the Sessional Programme for 1952-53. Members who may wish to offer papers during the coming Session are invited to communicate with the Secretary.

JOURNAL CASES AND BINDING, 1951

A binding case can be supplied for the twelve issues of the Journal, January-December, 1951 (Volume 29), price 11s., post free.

The price for binding volumes is 26s. per volume, inclusive. This price is for the half-leather binding which has been in use for some years.

It is requested that all parcels and Journals forwarded for binding should bear the name, address and rank of the member concerned. All volumes for binding must be despatched to the Institution by March 31st, 1952.

An Index will be included in all volumes bound. This Index will not be generally distributed, but members and others wishing to have a copy should apply to the Secretary.

THE MACLACHLAN LECTURE

General Conditions

Through the generosity of Mr. John MacLachlan (Retired Member), the Council was able in 1948 to institute an Annual Lecture to be competed for by Associate-Members. The conditions of the presentation are as follows :—

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture, and to be held annually.
2. The subject of the Lecture may be on any aspect of Structural Engineering so long as in every second year the subject shall be confined to steel structures.
3. Entrance into the competition for the Lecture shall be confined to Associate-Members of the Institution, who are under the age of 32 years.
4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.
5. No paper submitted shall have been published or read elsewhere.
6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s. 0d.
7. Should a competitor's paper be considered worthy of ranking second in merit, he will receive a consolation award of £5.
8. In the event of there being no winner of the competition in any one or more years, whether because no lecture is submitted or because no lecture submitted is considered to be of sufficient merit to warrant an award, or for any other reason, the Institution shall transfer these sums to the Research Fund of the Institution.

Particulars of the Competition for 1952

1. The MacLachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1952.
2. The subject of the Lecture shall be on any aspect of structural engineering.
3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture

in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

Lectures should be prepared in accordance with the requirements of the Literature Committee for publication in THE STRUCTURAL ENGINEER. Candidates may obtain a copy of these requirements on application to the Secretary.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Monday, March 31st, 1952.

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering ; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in the first category provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

The Colleges which have informed the Institution that courses in Structural Engineering are available are :—

- Belfast College of Technology.
- Birmingham Central Technical College.
- Bolton Municipal Technical College.
- Bradford Technical College.
- Derby Technical College.
- Dudley and Staffordshire Technical College.
- Glasgow Royal Technical College.
- City of Liverpool College of Building.
- L.C.C. Brixton School of Building.
- L.C.C. Hammersmith School of Building and Arts and Crafts.
- Manchester College of Technology.
- Middlesbrough Constantine Technical College.
- Salford Royal Technical College.
- South-West Essex Technical College, Walthamstow, E.17.
- Stockport College for Further Education.

Colleges in the second category provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete. The colleges which have informed the Institution that such courses are available are :—

- Brighton Technical College.
- Cardiff Technical College.
- Huddersfield Technical College.
- Leeds College of Technology.
- London Battersea Polytechnic.
- London Northampton Polytechnic.
- L.C.C. Westminster Technical College.
- Plymouth and Devonport Technical College.
- Preston Harris Institute.
- Wigan Mining and Technical College.
- Woolwich Polytechnic.

Students attending colleges in the first category are advised to take the organised courses in Structural Engineering. Students of Graduate Membership standard will usually be allowed to select subjects from courses provided by colleges in the second category.

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, on Tuesday, March 11th, when the Annual General Meeting will be held at 6.00 p.m., followed by a film on "The Moving of an Historic Building, Whitehall Gardens."

All members are urged to attend to elect the new Committee.

Hon. Secretary : D. B. Rogers, 4, Portland Rise, Finsbury Park, N.4.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Wednesday, March 12th, 1952

Joint Meeting with the Liverpool Engineering Society, at The Temple, 24, Dale Street, Liverpool, at 6 p.m. Mr. J. Cunningham, B.Sc., A.M.I.C.E., on "The Britannia Tubular Bridge over the Menai Straits."

Wednesday, March 19th, 1952

Dr. G. G. Meyerhof, M.Sc., A.M.I.C.E., F.G.S. (Associate-Member), on "Some Aspects of Soil Mechanics with reference to Foundations," at the College of Technology, Manchester, 6.30 p.m.

Tuesday, April 29th, 1952

Professor J. A. L. Matheson, M.B.E., M.Sc., Ph.D., M.I.C.E. (Member), on "Plasticity and Structural Design," at the College of Technology, Manchester, 6.30 p.m.

Thursday, May 15th, 1952

Annual Business Meeting.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 28, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Friday, March 28th, 1952

Mr. P. B. Morrice, B.Sc.(Eng.), on "The Research Station of the Cement and Concrete Association," at Stafford.

Tuesday, April 29th, 1952

Annual General Meeting at the James Watt Memorial Institute, Birmingham, 6 p.m., followed by a paper on "Piling in Engineering Construction," by Mr. J. Owen-Lake (Associate-Member).

Hon. Secretary : E. R. Deeley, A.M.I.Struct.E., Arranmoor, Adshead Road, Dudley, Worcs.

GRADUATES' AND STUDENTS' SECTION

Wednesday, April 30th, 1952

Mr. S. M. Cooper (Associate-Member), on "Investigation of the Failure of the Tacoma Narrows Bridge." The following films will be shown: "The Failure of the Tacoma Narrows Bridge" (Lond Version) ; "River to Cross" (wind tests on the Severn Bridge model). The meeting will be held at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 7 p.m.

Friday, May 30th, 1952

Short papers by members of the Section.

Hon. Secretary : M. H. Evans, B.Sc., 42, Church Hill Road, Handsworth, Birmingham,20.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, March 4th, 1952

Mr. G. S. Gowland (Associate-Member), on "Impressions of U.S.A. Welding Methods," at Middlesbrough.

Wednesday, March 5th, 1952

The above meeting will be repeated at Newcastle.

Wednesday, April 2nd, 1952

Annual General Meeting, at the Neville Hall, Newcastle, followed by a paper on "Data in the Drawing Office," by Mr. J. Ross.

All meetings will commence at 6.30 p.m., preceded by tea at 6 p.m.

Hon. Secretary : Ian MacGregor, M.I.Struct.E., 9, Ellison Place, Newcastle-upon-Tyne, 1.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, March 4th, 1952

Mr. H. M. Nelson, B.Sc., A.R.T.C., on "Plastic Design Applied to Structural Engineering," at the College of Technology, Belfast, 7.30 p.m.

Tuesday, April 22nd, 1952

Annual General Meeting, at the College of Technology, Belfast, at 7.30 p.m.

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Tuesday, March 11th, 1952

Mr. W. A. Fairhurst (Member), on "The Design of Engineering Structures, including Concrete Bridges."

Wednesday, April 17th, 1952

Annual General Meeting.

The above meetings will be held at the Ca'doro Restaurant, Glasgow, at 6 p.m.

Hon. Secretary : D. G. Drummond, B.Sc., M.I. Struct.E., A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10/12, Market Street, Torquay.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Tuesday, March 4th, 1952

Meeting at the South Wales Institute of Engineers, Cardiff, 6.30 p.m. Films will be shown.

Wednesday, March 5th, 1952

A meeting will be held at the Mackworth Hotel, Swansea, at 6.30 p.m., when the films referred to above will be repeated.

Friday, March 21st, 1952

A meeting will be held at Colwyn Bay.

Tuesday, April 1st, 1952

Students' Evening, at the South Wales Institute of Engineers, Cardiff, 6.30 p.m.

Friday, April 25th, 1952

Annual Dinner at the Osborne Hotel, Swansea.

Tuesday, May 6th, 1952

Annual General Meeting, at the South Wales Instiute of Engineers, Cardiff, 6.30 p.m.

Hon. Secretary : E. R. Steward, A.M.I.Struct.E., Edrom, Ashleigh Road, Blackpill, Swansea.

WESTERN COUNTIES BRANCH

The fourth meeting of the Session was held at Bristol University on Friday, January 4th, 1952, Professor A. G. Pugsley, O.B.E., D.Sc., M.I.C.E., M.I.Struct.E., F.R.Ae.S. (Branch Chairman), presiding.

A paper, entitled "The Fabrication of Steel Structures," was presented by Mr. O. H. Willey, which included comparisons between British and American practice. A keen discussion followed the paper.

The following meetings have been arranged :—

Friday, March 7th, 1952

Mr. J. A. Newton (Student), on "The Civil and Structural Engineers' Contribution towards the Reconstruction of Stapleton Road Gas Works, Bristol."

Friday, April 4th, 1952

Annual General Meeting, followed by Film Show.

Hon. Secretary : C. E. Saunders, M.I.Struct.E., Dunkery, Edward Road, Walton St. Mary, Clevedon, Somerset.

YORKSHIRE BRANCH

A Joint Meeting of the Yorkshire Branch and the Yorkshire Association of the Institution of Civil Engineers was held on Friday, January 11th, at the Blue Bell Hotel, Scunthorpe, at 6.30 p.m.

About 50 members and visitors attended and heard a paper on "The Construction of a Generating Station and its Sundry Erection Problems," by Mr. F. W. J. Saar, A.M.I.Mech.E., which was well illustrated with photographs taken during the construction of the Keadby Generating Station, near Scunthorpe.

A vote of thanks to the speaker was proposed by Mr. C. Cooper, M.I.C.E., and seconded by Mr. S. Richards, A.M.I.Struct.E.

The thanks of the Branch are due to Mr. Saunders, of the United Steel Structural Co., Ltd., for the help he kindly gave to make the meeting a great success.

A Meeting of the Yorkshire Branch was held on Wednesday, January 16th, in the Great Northern Hotel, Leeds, and was attended by 40 members and visitors.

A paper, entitled "Structural Engineering at Abbey Works," was presented by Mr. A. V. Hooker, A.M.I.C.E., A.M.I.Struct.E., and was excellently illustrated with lantern slides, and greatly appreciated by all present.

A vote of thanks to the speaker was proposed by Mr. Barlow, and seconded by Mr. Maddock, and carried with acclamation.

The following meetings have been arranged :—

Wednesday, March 19th, 1952

Mr. Hugh B. Sutherland, S.M. (Harvard), A.M.I.C.E., (Associate-Member), on "Problems in Foundation Engineering," at the Great Northern Hotel, Leeds, 6.30 p.m.

Wednesday, April 23rd, 1952

Annual General Meeting.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days, Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

Book Reviews

Structural Theory and Design, Vol. I, by J. McHardy Young, B.Sc., M.I.Struct.E., A.M.I.C.E. (London: Crosby, Lockwood). 288 pp. 25s.

The author presents in this edition the more elementary part of the subject, and Volume II will deal with more advanced theory and design for the specialist or practising engineer.

This first volume covers a wide range of theory from the study of materials and the design of simple beams to the deflection of frames, and part of the subject-matter is devoted to design in reinforced concrete. There are chapters on structural mechanics, normal structural analysis, deflections, fixed and continuous beams, rolling loads, columns, etc., and the author is to be congratulated on the neat and methodical layout of the whole of the work. The book is profusely illustrated, and each chapter ends with a series of examples selected from Associate-Membership examinations.

The student is encouraged throughout the book to make comparisons by means of alternative methods of analysis, and stress is laid on the fact that designers must acquire the skill to be able to modify theoretical considerations to suit the needs of practical design.

The book achieves its object with great success, and will prove of assistance to students preparing for the examination of this Institution.

D. T. W.

Structural Theory and Design, Vol. II, by J. McHardy Young, B.Sc., M.I.Struct.E., A.M.I.C.E. (London : Crosby Lockwood). 300 pp. 25s.

Volume II of this excellent production was awaited with interest following the general praise with which the first volume was received.

The book is intended to cover the theory of structures paper in the Associate-Membership examination of this Institution, and deals widely with redundant framed structures, arches, portals and rigid frames.

There are most informative chapters on the analysis and design of building frames, earth pressure, soil mechanics, foundations, etc., and the high standard reached in the previous volume is maintained throughout the second contribution.

One unusual and welcome chapter is devoted to miscellaneous problems in steelwork and reinforced concrete, nor has the question of connections been forgotten—in fact the final chapter deals exclusively with the subject—the author pointing out that "The design of any structure cannot be efficient unless the connexion between the individual members is designed with the same care as the members themselves," and the concluding chapter covers structural connections in steel, reinforced concrete and timber.

Many of the numerical examples are based on the final examination papers set by this Institution, and candidates for this examination will find these numerous and varied in type.

The author is once more to be congratulated on the general layout and on the clarity and simplicity of the diagrams and illustrations, and students and practising engineers alike will welcome this second volume as warmly as they did the first.

D. T. W.

An Introduction to Experimental Stress Analysis, by G. H. Lee. (London : Chapman & Hall,; New York, Wiley.) 319 pp., 9 in. × 6 in. 44s.

For many of the problems confronting the structural engineer to-day the classical methods of analysis have been found inadequate, and the experimental determination of stresses is being used to an increasing extent in the solution of such problems. The present book by G. H. Lee is a concise and useful collection of the fundamentals of the experimental approach. The book deals with the essential mathematical relationships required in the analysis of strain and describes a variety of ways in which strain may be measured.

The methods described for the analysis of strain gauge rosettes are cumbersome, and the author is apparently unfamiliar with the methods developed in this country.

The electric strain gauge is, not unnaturally, given a great deal of attention. Techniques for both static and dynamic tests are described. The problem of zero drift of electric resistance gauges is not discussed, although from the point of view of the structural engineer this is one of their chief drawbacks.

The photoelastic approach is adequately described and a separate chapter is devoted to the Brittle Lacquer method. The mechanical strain gauges described are almost exclusively American, and not in common use in this country. It is unfortunate that only one brief paragraph is given on the acoustic gauge, although gauges of this type have been largely used for structural

research in this country, particularly by the Building Research Station, and earlier by the Steel Structures Research Committee.

The book is set clearly and is well illustrated. It should form a useful source of reference for workers engaged in Experimental Stress Analysis.

F. B. B.

Molesworth's Handbook of Engineering Formulæ and Data. (London : E. & F. N. Spon, 1951.) 1,686 pp., 6½ in. × 4 in. 900 illustrations, numerous tables. 32s. 6d.

In the 34th edition of this well-known and useful book, the text has been completely re-written and substantially enlarged to bring it entirely up-to-date, and it has been re-illustrated. All the fundamental data included in the earlier editions is, however, retained.

The book is divided under four headings : General, Civil and General Engineering, Mechanical Engineering and Electrical Engineering. Some of the sections included under Civil and General Engineering are the following :—Structures and Bridges, Welding, Concrete and Reinforced Concrete, Piles, Breakwaters and Marine Works, Soil Mechanics and Earthworks, and Building Construction.

Moving Forms, by L. E. Hunter. (London : Concrete Publications, 1951. 56 pp., 9 in. × 16 in. 7s. 6d.)

The author makes a very interesting and useful contribution to a form of construction about which very little is written in British text-books.

For those with knowledge and experience of the work, this book provides a check on innumerable details. To those with no previous experience about to embark upon a construction operation of this type, the book is very informative.

The many excellent photographs add considerably to the usefulness and interest of this publication but I think the line diagrams might have been a little more in keeping with the text and photographs.

The author only deals very briefly with the work of dismantling the roof shuttering and the wall forms. This part of the work is quite a major operation and some enlargement on this subject would have been an asset.

There is of course no substitute for the excitement and anxiety of carrying out a sliding form contract, particularly where large numbers of jacks are involved.

The Appendix covering the hints and reminders is a very useful addition indeed and however much anyone may think they know about sliding form jobs, it would be advantageous just to read through this Appendix before commencing work.

I notice in the chapter dealing with shutters travelling horizontally that the author appears to have some unorthodox ideas on this type of work.

G. B. R. P.

Modern Bridge Construction : "Treatise setting forth the Elements of Bridge Design and Illustrating Modern Methods of Construction," by F. Johnstone Taylor. Second edition, revised and enlarged. (London : Technical Press, 1951.) 331 pp., 8½ in. × 5½ in., illustrated. 30s.

This book of 330 pages provides interesting and instructive reading on the principles of Bridge Engineering, and describes design and construction of bridges in masonry, steel and reinforced concrete, and includes swing and bascule types both superstructure and foundations. It omits reference to such modern materials as prestressed concrete and lightweight alloys.

In accordance with the preface to the first edition of 1930, which is republished in this second edition, the aim is to deal with the subject in a clear and concise manner so as to serve the needs of the average civil engineer and provide a grounding in the subject for engineering students. This aim, it may be stated, has been carried out with considerable success and the value of the work is enhanced by the many references to published accounts of the structures described, thus rendering further study feasible.

The writer of this review however, would not be completely in agreement with some of the statements made, such as :—

p. 9— Safe intensity of stress, which should not be exceeded, 3 tons per square foot for good concrete.

p. 13— Grosvenor Bridge at Chester, appears to be a "segmental" arch of 200 ft. span, not "semi-circular," as stated in both the 1930 and 1951 editions.

p. 193—Old Waterloo Bridge is alluded to as an "ill-fated structure." As the old bridge stood successfully for over a century, in spite of the varying conditions of the river-bed caused by the removal of Old London Bridge subsequent to its construction, it would appear to the reviewer to have served its purpose admirably.

p. 255—There is some want of clarity in the relative constructional depths in reinforced concrete bridges of simple spans as compared with framed spans where the former is described as 1/50 of span and the latter as 1/35 to 1/25.

p. 267—The subject of joint filling in "plastic" and "elastic" cements requires some further amplification.

p. 287—Under "Recent Developments," it is suggested that reinforced concrete bridge-work had reached a final stage of development—it would appear to the reviewer that, with the adaptation of "prestressed" concrete, the design of all ordinary type bridges is likely to undergo a radical change.

p 288—The reviewer would question the statement which classifies the new Waterloo Bridge as being of the arch type like those at Chiswick and Hampton Court.

p. 289—In a 1951 book a statement is made that Traneberg Bridge at Stockholm of 563 ft. span, is believed at the present to be the longest span concrete arch yet constructed.

p. 291—The references to the uses of aluminous cements would require further amplification.

R. P. M.

Concrete Block Construction for Home and Farm, by J. R. Dalzell and G. Townsend. (Chicago : American Technical Soc., 1951 ; London : Technical Press. 216 pp., 8½ in. × 5½ in. 22s.)

It is stated in the preface that the authors are convinced that almost anyone can build a structure from concrete blocks and there is no doubt that many with no previous knowledge or experience will be able to find in this book many of the essentials of building construction with particular reference to the use of hollow concrete blocks and much guidance on practical points of construction which are so important to the novice. The book will be of value to farmers, landowners, handymen and all others who wish to construct a small building such as a house, a barn or cowhouse, with the direct

employment of labour and who do not wish to call upon the services of an architect or builder.

There are good chapters on the basic materials and on the characteristics and mixing of concrete. In general, the essentials of sound construction and proper use of materials are well brought out, although it may be mentioned that the importance of level filling of gauge boxes is not emphasised and indeed the photograph showing a box about to be emptied indicates aggregate piled up in the box and not struck off to a level surface. Nor is any guidance given as to a means of measuring bulking of sand, although this can be done with a very simple test and is to be preferred in the reviewer's opinion to the suggested tests for the feel of sand having various degrees of moisture content.

The chapter on formwork is very thorough and the illustrations are very clearly drawn and easily followed, but it must be said that the formwork shown in these illustrations is a very superior article and it is doubted whether the amateur builder or even professional builder for that matter, would be prepared to produce such high-class formwork for foundations and other parts of a small structure in this country.

The theory and design of footings is dealt with at some length and there is much useful information here, but three points of criticism may be made as follows :—

The settlement of structures on clay is attributed to failure of the soil to support the moisture load and it is suggested that by using a footing of suitable width all settlement can be eliminated, whereas in fact some settlement is bound to occur however adequate the footings. Also, the depth of penetration of frost is made one of the principal criteria of depth of footings below the surface of the ground, and no mention is made of the importance of going below the level of seasonal variations in moisture content of the soil which in general is much more important in this country than the question of frost penetration, more particularly in the case of light structures such as those under consideration. A point of detail which is a little puzzling is the provision of a small V-shaped key between footings and foundation and which appears to be so small as to be of little value and hardly worthy of the trouble of forming it. Another point not mentioned in dealing with foundations is the importance of preparing the surface of the soil immediately before depositing the concrete for the footings, indeed, in the section on cold weather concreting, it is suggested that frozen ground should be thawed out by fires built over it. If concrete were then deposited there is considerable risk of settlement due to the disintegration of the surface of the soil by successive freezing and rapid thawing with the liberation of moisture.

It is further noted that the effect of freezing of concrete is explained as a cessation of reaction between cement and mixing water, and it is suggested that if the newly-placed concrete becomes frozen it should be thawed out slowly. There is, however, a grave risk that the freezing of concrete may not only stop setting and hardening but cause disintegration of the material and, more seriously, the unseen internal disruption so that it will never attain anything approaching its normal strength, and it is felt that the reader should be warned against the acceptance of any such concrete without proper examination.

Those who expect to find a book devoted entirely to concrete block construction will be disappointed since only one-third of the book deals specifically with concrete blocks and the various details of their use.

The use of semi-dry mixes in the manufacture of concrete blocks does not appear to be dealt with at all adequately, neither is the technique of machinery mad concrete blocks described.

It may also be pointed out that the book is a American publication, and relates more particularly t American practice and standards of block manufactur and construction.

C. W. G.

Design of Sheet Pile Walls (Berechnung mehrfac gestutzter Spundwande), by E. Lackner. (W. Ernst Berlin, 1950.) 64 pp. + xi ill. DM. 6.

After a brief summary of current methods of designin sheet pile walls, the author outlines an analytical an graphic procedure for walls with free and fixed eart support ; both contillor and anchored walls with support at two levels are considered. The method used is base on the customary analogue of a continuous beam wit flexible supports ; the loading on this beam is given b the active earth pressure (behind the wall) derived fro Coulomb's method, which is also used to estimate th passive pressure (in front of the wall).

Recent observations on model walls have shown tha the passive pressure is not fully unobserved except i special cases and that Coulomb's method gives therefor results on the unsafe side. Since for normal penetra tions of sheet pile walls, the resulting error is conside able, more advanced methods of calculation have to b used.

G. G. M.

Progress in Metal Physics 2. Edited by Bruc Chalmers. (London : Butterworths' Scientific Publica tions, 1950.) Pp. viii, plus 213, 10 in. × 6 in., illus. 45

This is the second of an annual series of volumes th purpose of which is to present authoritative reviews o the present state of knowledge in specialised aspect of metallurgy and metal physics, and particularl covering work in which significant progress has bee made.

The present volume contains chapters on orde disorder changes in alloys, rate processes in physica metallurgy, anisotropy in metals, developments i magnesium alloys, and a symposium on polygonisatio Each section contains a comprehensive bibliography.

The book is clearly printed, well produced an illustrated.

Estimating for Building and Civil Engineerin Works, by Spence Geddes. (London : George Newne 1951.) 472 pp., 9¾ in. × 7½ in. 63s.

This is a comprehensive book on estimating an tendering for building and civil engineering works c construction.

In the first part, the different methods of tenderin and of compiling estimates are given, including th "Bill of Quantities," "Schedule of Rates" and "Cos Plus" forms of tender.

From the detailed data given it is possible to estimat the cost of work carried out by modern methods c construction using the latest types and sizes of moder plant, and also for work carried out by hand.

The various trade sections, numbering eightee devoted to detailed information required for estimatin administration and balance of labour for individu trades employed on the site, are given in alphabetic order for easy reference.

Two sections covering weight of materials and usef tables complete this valuable book of reference for civ engineers, surveyors, building and civil engineerin contractors.

An Investigation of the Behaviour of a Riveted Plate Girder under Load*

By S. Mackey, M.E., Ph.D., A.M.I.C.E., A.M.I.Struct.E.
and D. M. Brotton, B.Sc., Ph.D. (Graduate)

Introduction

In an earlier paper[1] the authors described tests which they had carried out on a welded mild-steel plate girder, the overall dimensions of which were 20 ft. 9 in. long by 3 ft. 1½ in. deep. This was a quarter-scale model of 12 ft. deep crane runway girders for the new Margam Steel plant at South Wales, and formed part of an investigation into the stress distribution in mild steel plate girders involving three further girders of similar size which have since been tested; two of welded and one of riveted construction.

Since riveting is widely employed in the fabrication of structural steelwork it is felt that a description of the behaviour of the riveted test girder under load will be of interest to structural engineers. The riveted girder was designed primarily to obtain information on web and stiffener stresses; hence direct comparison with the Model Margam girder in many respects is difficult, but where any differences or similarities in the behaviour of the two occur they will be discussed in the present paper.

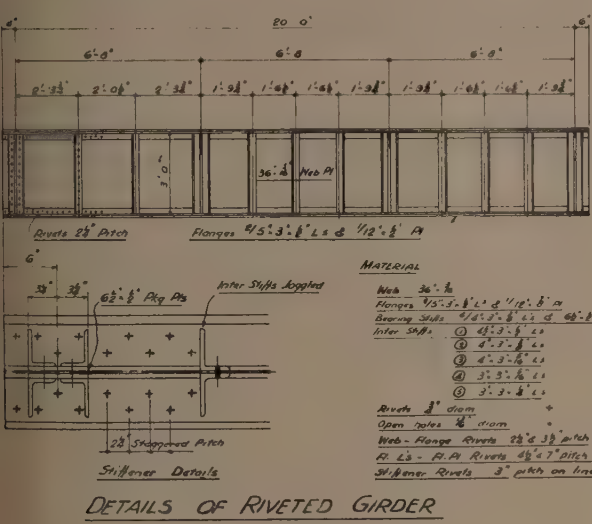


Fig. 1

Design and Details of Test Girder

Information was desired on web panels subjected to bending stresses, shear stresses and a combination of both, and thus, a "two-point" loading system was adopted, equal loads being applied at the third points of the span.

The girder was of normal riveted construction as shown in Fig. 1. As critical conditions were desired primarily in the web panels, the web plate was made 3/16 in. thick, which was considered a practical minimum. To prevent flange yielding causing premature failure, the flanges were designed to carry a load approximately 10 per cent. in excess of the load estimated to cause the web plate to yield in shear.

Five complete sets of intermediate stiffeners were provided loose, to be bolted to the girder as required. All these, which varied from 4½ in. × 3 in. × ½ in. angles to 3 in. × 3 in. × ¼ in. angles, were joggled over the vertical legs of the flange angles and fitted. Ordinary black bolts were used for their connection to the web.

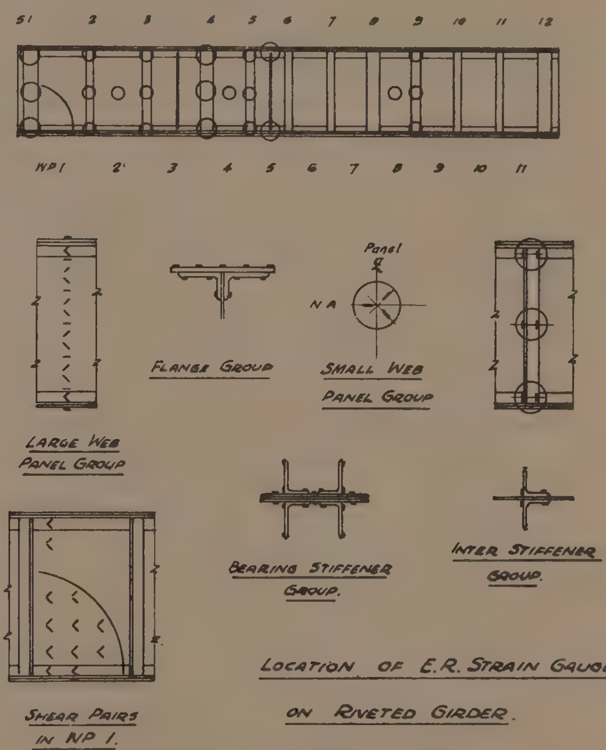


Fig. 2

General Description of Tests

As in the case of the earlier girder, the tests were carried out in the test house at Messrs. Dorman Long and Co., Ltd., Britannia Works, Middlesbrough, using the 1,250 ton horizontal type Avery testing machine. A description of the testing machine, together with the method of application of load and general testing procedure is given in the previous paper[1].

The riveted girder was received from the fabricating shops with the flange angles and flange plates riveted up completely, except for the open holes left in the vertical legs of the flange angles for fixing the intermediate stiffeners. The end and load-bearing stiffeners were completely riveted up.

The tests were divided into five sections, each appertaining to one particular set of intermediate stiffeners.

*Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, April 24th, 1952, at 6 p.m.

For ease of reference, numbers have been applied to the various sizes of intermediate stiffeners as follows :—

Stiffeners (1)—4½ in. × 3 in. × ½ in. angles
,, (2)—4 in. × 3 in. × ⅜ in. ,,
,, (3)—4 in. × 3 in. × 5/16 in. ,,
,, (4)—3 in. × 3 in. × 5/16 in. ,,
,, (5)—3 in. × 3 in. × ¼ in. ,,

The disposition of the electric resistance strain gauges on the girder is shown in Fig. 2. Approximately 300 active gauges were used in the test.

Properties of Material and Girder Cross-Section

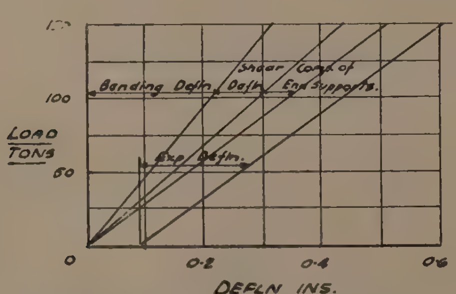
To determine the properties of the web and flange material, test pieces were cut from short additional lengths of the same plates and tensile tests carried out on these. Mean results of two tests on each plate, together with the calculated properties of the girder cross-section are shown in Table 1. The value given in this table for the torsion constant J was obtained by relaxation methods as described elsewhere[2].

TABLE 1.—Properties of Material and Girder Cross-Section

Gross cross-sectional area		
Moments of Inertia	I_{xx}	= 8196 in.^{4}
	I_{yy}	= 316.4 in.^{4}
Section Moduli	Z_{xx}	= 443.0 in.^{3}
	Z_{yy}	= 52.7 in.^{3}
Radii of Gyration	r_{xx}	= 15.59 in.
	r_{yy}	= 3.06 in.
Torsion Constant	J	= 9.14 in.^{4}
Modulus of Elasticity	E	= 13,450 tons/sq. in.
Modulus of Rigidity	C	= 5,170 tons/sq. in.
Yield stress—Web		= 21.50 tons/sq. in.
Comp. Fl.		= 17.62 tons/sq. in.
Tension Fl.		= 16.95 tons/sq. in.
Maximum stress—		
Web		= 28.50 tons/sq. in.
Comp. Fl.		= 28.90 tons/sq. in.
Tension Fl.		= 29.30 tons/sq. in.

Girder Central Deflection

The girder was tested as a simply-supported beam on a span of 20 ft. and deflection readings were taken at the centre of the span, and close to the end supports, for each set of stiffeners. Fig. 3 shows the graphs of



CENTRAL DEFLN FOR RIVETED GIRDER.

Fig. 3

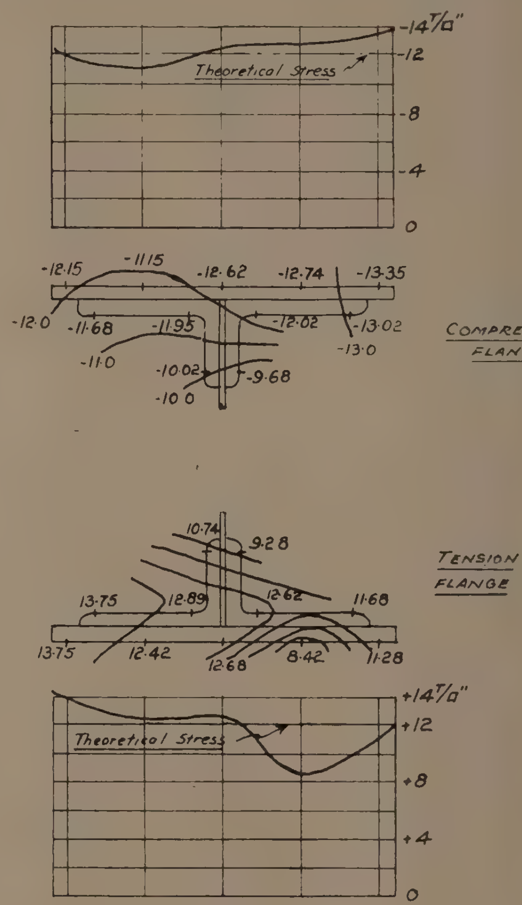
theoretical bending and shear deflection. Superimposed on the figure is the experimental deflection for the girder with stiffeners (2) fitted, together with the corrections necessary to allow for compression of the end supporting packings.

Values of the theoretical deflections at 150 tons load are given below :—

Theoretical bending deflection	0.315 in.
Theoretical shear deflection	0.120 in.
Theoretical total deflection	0.435 in.

From these it can be seen that bending accounts fo[r] 72.5 per cent. of the theoretical deflection, the remaining 27.5 per cent. being due to shear.

Table 2 shows values of the experimental central girde[r] deflection obtained with the various sizes of intermediate stiffeners employed. These correspond to a total applie[d] load of 150 tons and are corrected to allow for com[-]pression of the end supports. The percentage differenc[e] of the experimental deflection from the correspondin[g] theoretical value, shown in Table 2, is small in ever[y] case and may be due to experimental error. From the fact that in four out of the five cases examined the



FLANGE STRESS DISTRIBUTION FOR GIRDER AT 150T LOAD WITH STIFFENERS (5)

Fig. 4

experimental deflection is greater than the theoretica[l] value, one might infer that the girder was not behavin[g] completely as an integral unit. The differences howeve[r] are insignificant as far as practical design is concerned.

TABLE 2

Intermediate Stiffeners	(1)	(2)	(3)	(4)	(5)
Experimental Deflection... ...	0.467	0.442	0.449	0.434	0.464
% Difference from theory	+7.4	+1.6	+3.2	—0.2	+6.7

Flange Stresses

From readings of strain gauges placed around th[e] flanges the stress distribution across the latter wa[s] obtained. The different sets of intermediate stiffene[rs]

do not materially affect the distribution and that shown in Fig. 4, which is drawn for the girder with stiffeners (5) fitted, is typical of the results obtained in the other cases. The stress distribution across the compression flange is fairly uniform but in the tension flange a distinct reduction in the experimental bending stress occurs half-way across the top side of the girder flange as tested. This would appear to indicate a fault in the strain gauge placed at this point but additional groups of strain gauges placed around the flange close to the

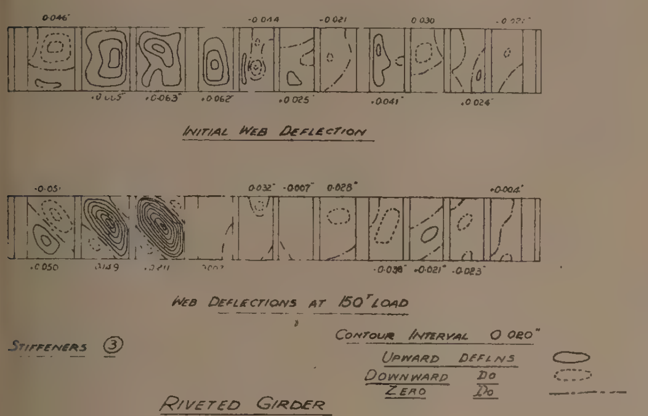


Fig. 5

section under consideration verify this local reduction in stress. It may be noted that a similar but less marked reduction occurs on the lower half of the compression flange as tested. The general trend of the experimental results seems to indicate the presence of a small amount of lateral bending on the girder. If this is accepted and a mean linear distribution through the experimental points is taken then the effects of the stress variations is far less marked.

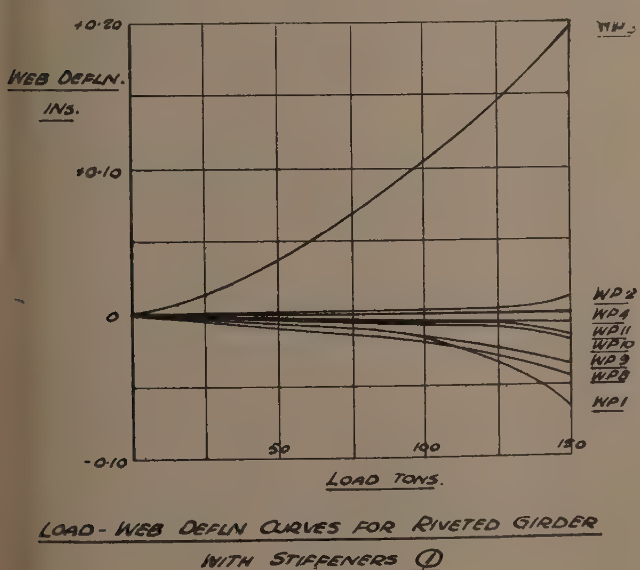


Fig. 6

As can be seen from Fig. 4, good agreement is obtained between the theoretical extreme fibre bending stress calculated on the gross moment of inertia and the mean of the experimental results.

Web Lateral Deflections

Contours of initial web deflection and web deflection under load were obtained for all stiffener groups. From these readings Table 3 has been constructed giving the initial deflections and deflections under load of particular web panels supported at the edges by different intermediate stiffeners. Fig. 5 shows contours of web deflection for the girder with stiffeners (3) fitted, at a load of 150 tons. In panel WP1 the figure shows the panel divided into two half-waves of approximately equal depth and opposite in direction. In most of the other shear panels where the deflections were sufficiently

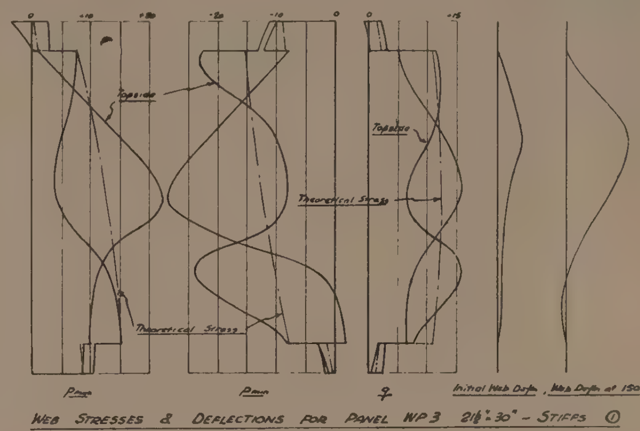


Fig. 7

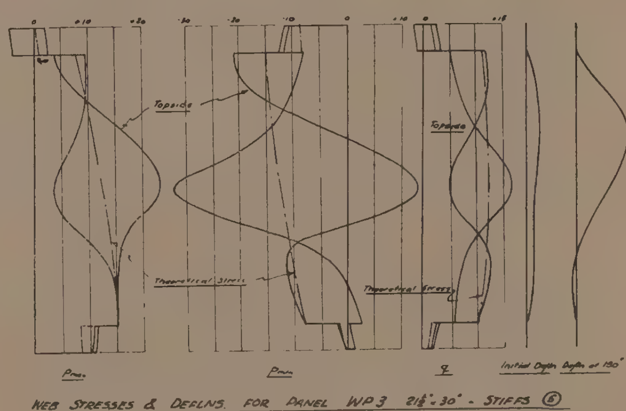


Fig. 8

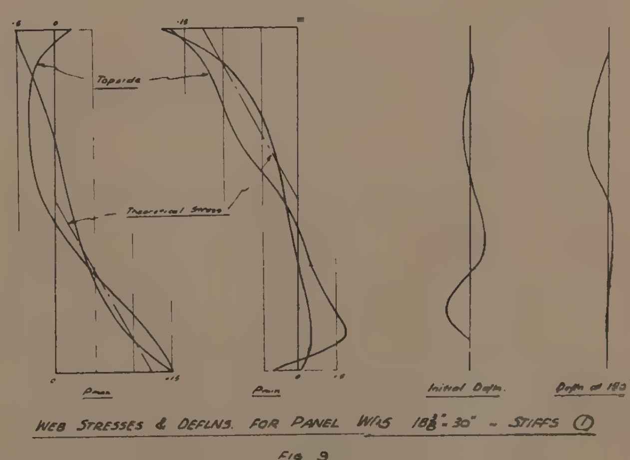


Fig. 9

large to adopt a characteristic shape, one large buckle occurred across the compression diagonal and two small buckles in diagonally opposite corners. Similar shapes of deflection contours were obtained with the other stiffener types.

In all cases the buckles showed no tendency to spread from one panel to another for loads up to the theoretical critical values for the web panels in question. It is apparent therefore, that even the lightest intermediate

stiffeners used in the test were adequate to retrain the girder web. The effect of initial deflections on the shape of the deflection contours under load was not pronounced, although in cases where the former were large they did, however, decide the direction of the deflections under load.

TABLE 3

Type, Size	Panel No.	Stiffeners	Initial Deflection in ins.	Deflection at 150 tons in ins.
Shear Panel 26″ × 36″	WP 1	(1)	+0.041	—0.104
		(2)	—0.042	—0.062
		(3)	—0.046	—0.051
		(4)	—0.042	—0.078
		(5)	—0.046	—0.103
	WP 2	(1)	+0.047	+0.084
		(2)	+0.067	+0.129
		(3)	+0.065	+0.149
		(4)	+0.073	+0.040
		(5)	+0.049	+0.102
	WP 3	(1)	+0.081	+0.216
		(2)	+0.053	+0.195
		(3)	+0.063	+0.211
		(4)	+0.075	+0.209
		(5)	+0.084	+0.248
Shear Panel 20″ × 36″	WP 8	(1)	+0.035	—0.064
		(2)	+0.037	—0.069
		(3)	+0.041	—0.038
		(4)	+0.040	—0.049
		(5)	+0.024	—0.052
	WP 9	(1)	—0.035	—0.051
		(2)	—0.032	+0.022
		(3)	—0.030	+0.020
		(4)	+0.028	+0.015
		(5)	+0.017	+0.026
	WP 10	(1)	—0.016	—0.023
		(2)	+0.015	—0.018
		(3)	+0.024	—0.023
		(4)	—0.019	+0.043
		(5)	—0.027	—0.041
	WP 11	(1)	—0.029	—0.015
		(2)	—0.009	+0.019
		(3)	—0.021	+0.004
		(4)	—0.022	+0.028
		(5)	—0.025	—0.014
Bending Panel 20″ × 36″	WP 4	(1)	+0.046	—0.015
		(2)	+0.038	—0.006
		(3)	+0.062	—0.008
		(4)	+0.041	—0.007
		(5)	+0.053	—0.005
	WP 5	(1)	+0.024	—0.011
		(2)	+0.031	—0.003
		(3)	—0.044	—0.032
		(4)	—0.039	—0.006
		(5)	—0.017	+0.017
	WP 6	(1)	+0.022	—0.010
		(2)	+0.042	+0.013
		(3)	+0.025	—0.007
		(4)	+0.044	—0.007
		(5)	+0.024	—0.017
	WP 7	(1)	+0.005	—0.023
		(2)	—0.015	—0.020
		(3)	—0.021	—0.028
		(4)	—0.016	—0.046
		(5)	+0.022	—0.017

From Table 3 it can be seen that the initial deflections were usually a little higher in panels WP1 to WP3 than in panels WP8 to WP11, which had a shorter length. The largest initial deflections, however, were about 1/450th of the clear dimension between stiffeners. The differences in magnitude of the deflections caused by fitting different sets of stiffeners to the girder were irregular ; in cases where the configuration of the panel deflections remained almost unchanged, the initial deflections also remained unchanged. No regular relation existed between depth of buckles under load and rigidity of the intermediate stiffeners. Very little variation was noted in those web panels having large deflections, but where the web deflections were comparatively small, considerable variation in the depth of buckles occurred.

In considering the deflections under load it was found that the values obtained were higher for panels WP1 to WP3 than for the shear panels of shorter length. Generally the ratio of deflection/clear panel length ranged from about 1/200 at a load corresponding to 1.3 times the Timoshenko buckling value, to about 1/1000 at 0.8 times the latter load. At the actual Timoshenko load the ratio was approximately 1/350.

Web deflections for bending panels were much smaller than for shear panels of the same size and were generally less than 1/1000 of the clear panel length at 0.9 times the Timoshenko buckling load.

Web Stresses

(a) Shear Panels

Distributions of maximum and minimum principal stress and shear stress were obtained for panel WP3.

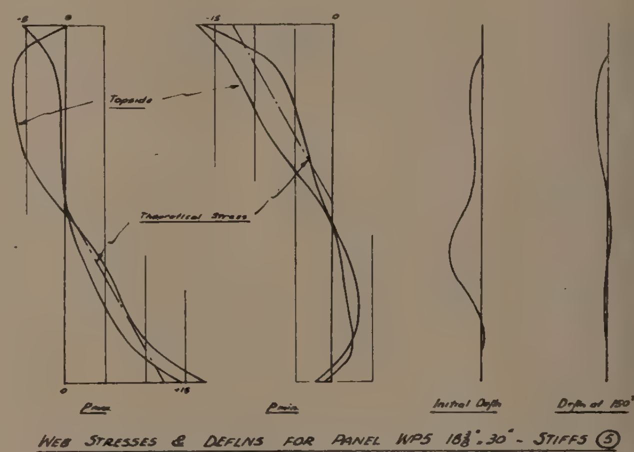


Fig. 10

These distributions, together with profiles of initial web deflection and deflection under load are shown in Figs. 7 and 8 for the girder with stiffeners (1) and (5) respectively. Similar diagrams were obtained with the other sets of stiffeners in place.

The positions of the points of inflexion on the web deflection profiles under load agree with the location of points of zero bending stress on the stress distribution curves. Although plate bending stresses at the toes of the flange angles were quite large the web deflection profile tends to justify the assumption of simply supported edge conditions for the web plate. Theoretical middle surface stresses calculated on this assumption agree fairly well with the experimental values.

TABLE 4

Intermediate Stiffeners No.	p max.		p min.		q	
	Top	Under	Top	Under	Top	Under
(1)	+22.20	+4.00	—8.50	—28.8	+6.30	+15.5
(2)	+19.50	+2.40	+6.0	—31.9	+6.10	+15.5
(3)	+23.40	+3.30	+13.30	—30.2	+4.50	+15.90
(4)	+21.80	+5.20	+10.20	—25.4	+5.10	+14.10
(5)	+23.30	+3.50	+12.70	—32.2	+5.00	+16.70

Table 4 shows values of principal and shear stresses in panel WP3 for different intermediate stiffeners. All the stresses are plotted for a girder load of 150 tons, which is 1.3 times the Timoshenko buckling load.

The magnitude of the values in this table indicates that yielding of the web material must have occurred, but Figs. 7 and 8 show that this has been only of a local character on one surface of the plate and that at no point of the web has the middle surface stress exceeded the yield value.

(b) Bending Panels

Diagrams of principal stress and web deflection in panel WP5 with stiffeners (1) and (5) are shown in Figs. 9 and 10 respectively. The largest bending stresses occurred near the compression flange where the buckles were deeper, and the agreement between points of zero bending stress and points of inflexion was quite reasonable. As in the case of the shear panels, bending stresses occurred at the toes of the flange angles whereas simply-supported edge conditions were indicated by the web deflection profile.

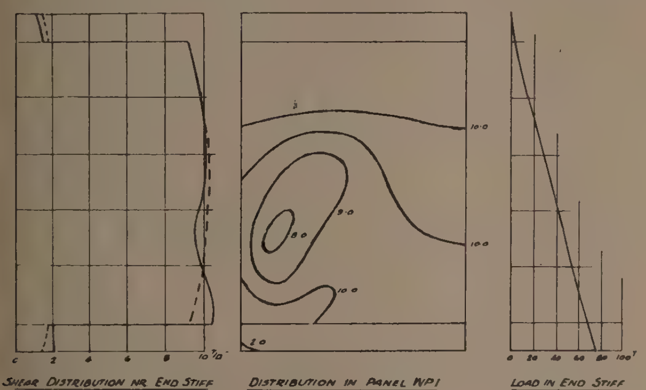


Fig. 11

Table 5 gives values of principal stress in panel WP5 at a point 6 in. from the toes of the flange angles. The stress values given are for a girder load of 150 tons, which corresponds to 0.9 × the Timoshenko buckling load for the web panel. Buckling in the direction of the compressive principal stress was generally larger than in the direction of the tensile principal stress but the effect of intermediate stiffeners on the buckling or bending stresses was irregular and very small compared with their effect on shear panels. Agreement between the middle surface stresses and the theoretical stresses is quite good.

TABLE 5

Stiffeners	p max.		p min.	
	Top	Under	Top	Under
(1)	—3.30	—0.80	—9.50	—4.90
(2)	—2.00	—1.20	—8.00	—2.70
(3)	—2.30	—0.60	—8.70	—5.10
(4)	—2.50	—2.50	—9.50	—5.90
(5)	—6.50	—0.40	—9.70	—4.70

Shear Stress Distribution in Panel WP1

"Shear pairs" of strain gauges in the end panel yielded the shear stress distribution shown in Fig. 11 for a girder load of 150 tons. The shear stress in the immediate vicinity of the end reaction was greater than the theoretical value and this increase extended beyond the flange angles into the web plate. The stress, however, soon began to fall and reached its theoretical value within approximately 6 in. from the toe of the flange angles. The stress continued to decrease and a region of reduced shear stress was centred at a point approximately 12 in. from the tension flange and 4 in. from the end stiffener. Beyond this point the stress increased, gradually approaching its theoretical value, which was reached within 6 in. of the toes of the compression flange angles. In the vertical legs of the compression flange angles the shear stress was only slightly below the theoretical value.

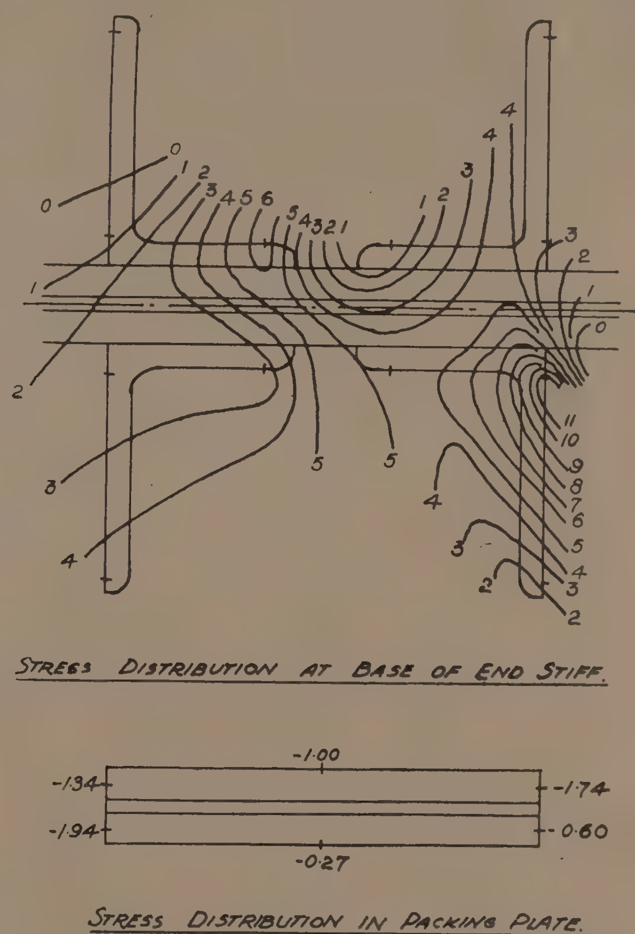


Fig. 12

Along the length of the girder, the shear stress assumed normal distribution at a distance of 20 in. approximately from the end support, which was just before the first intermediate stiffener S2 was reached. This marked the extremity of the zone of influence of radial stresses due to the concentrated load of the end reaction.

Bearing Stiffeners

The curve of axial load in the end stiffener shown in Fig. 11 was obtained from the shear stress distribution diagram of the same figure. The transmission of load to the web plate was almost uniform over the whole depth of the girder, and a difference between the theoretical and measured stiffener loads of 1.3 tons was probably due to experimental error.

Fig. 12 shows the distribution of axial stress in the end stiffener angles and the vertical legs of the flange angles

at a girder load of 150 tons, giving an end reaction of 75 tons. The stress distribution was very irregular, varying from a compressive stress concentration of over 11.0 tons per sq. in. near the heel of one angle to a small tensile stress in part of the outstanding leg of the diagonally opposite angle.

By integrating the distribution diagram, a figure of 35 tons was obtained for the load carried by the stiffener angles alone, which was equivalent to a mean stress of 3.52 tons per sq. in. over the gross area of the angles. The obvious conclusion from this was that the remaining 40 tons was transmitted directly to the web plate and flange angles. The high proportion of load transmitted to the web was probably due to the comparatively narrow bearing plate placed between the two inner rows of rivets through the flange plate and angles. Strain gauges placed at the centre and top of the stiffener angles indicated a more uniform distribution and gave values of 24.9 tons and 6.75 tons respectively for the axial loads in the angles. Thus load transfer from the angles along their length was fairly uniform, being low at first and then more rapid as the compression flange was approached.

Strain gauges were also fixed to the stiffener packing plates at points 2 in. from the ends, near the tension flange. All these gauges indicated the presence of compressive stresses, but they were low, the values corresponding to a girder load of 150 tons being shown on Fig. 12.

Figures somewhat similar to those for the end stiffeners were obtained for the loading stiffener S4.

Intermediate Stiffeners

The stresses in intermediate stiffener S3 corresponding to a girder load of 150 tons are given in Fig. 13 for the

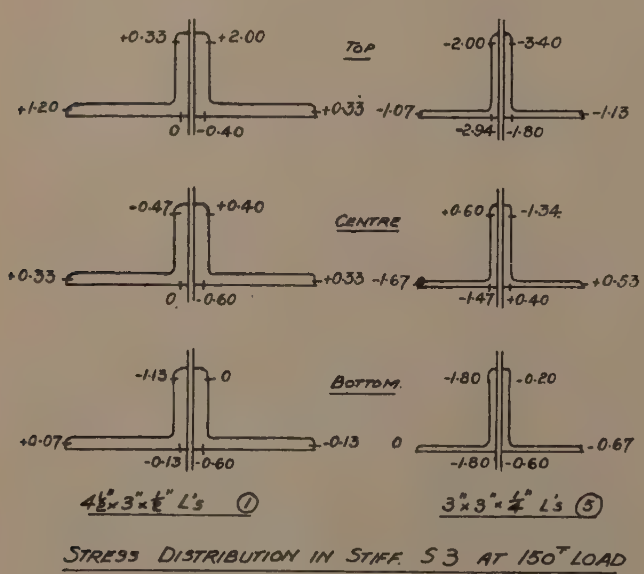


Fig. 13

heavy angles (1) and the light angles (5). This girder load was considered the highest which it was safe to repeat in view of the local yielding in panel WP3, mentioned earlier. In the $4\frac{1}{2}$ in. $\times$ 3 in. $\times$ $\frac{1}{2}$ in. angle stiffener S3 tensile stresses were recorded mainly at the top, under a girder load of 150 tons, and there was a marked increase in compressive stress as the tension flange was approached. At no place, however, did the stiffener stress reach a high value. Almost all the gauges on the 3 in. $\times$ 3 in. $\times$ $\frac{1}{4}$ in. angle stiffener S3 indicated compression, the highest recorded value obtained near the compression flange reaching a value of 3.4 tons per sq. in. Stresses were therefore higher in the lighter stiffeners, but up to 150 tons girder load, corresponding to 1.3 times the Timoshenko buckling load for the adjacent panel WP3, the stresses were still very low.

Crippling Run

In view of the small stresses recorded in the intermediate stiffeners at the previous maximum girder load,

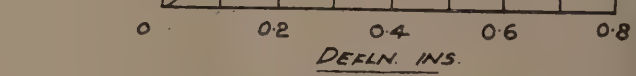


Fig. 14

stiffeners (5) were retained on the girder for the final run of load.

The load-central deflection curve for the girder during the crippling run is shown in Fig. 14. The deflection was proportional to the load up to the previously applied maximum of 150 tons. Thereafter, it increased at a more rapidly increasing rate until a load of 175 tons was reached, when it was noted that the deflection began to

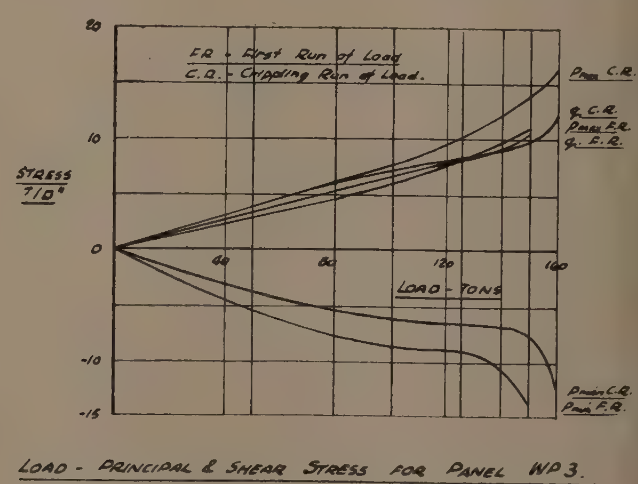


Fig. 15

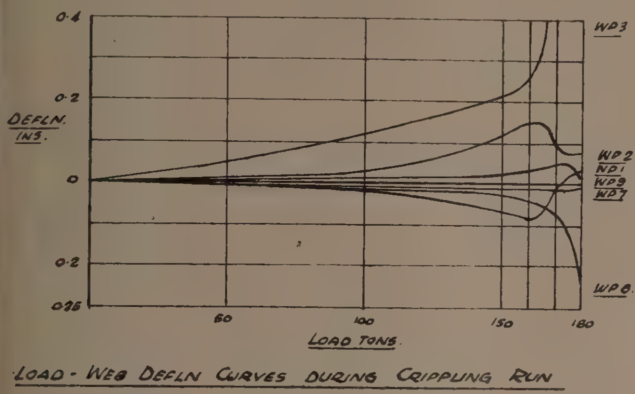
increase slowly under constant load. At this stage the dials were removed and no further deflection readings were taken.

The group of strain gauges placed around the flanges indicated no yielding of the flange material up to the maximum load at which readings were possible.

Fig. 15 shows graphs of middle surface values of principal and shear stresses in panel WP3 taken during the first run of load on the girder and also on the crippling

run. In the first run of load the curve of P min. shows an increase in the rate of increase of stress after a girder load of 130 tons has been reached, indicating the presence of local surface yielding in the panel. If the middle surface values of stress at 130 tons load are substituted in the equation giving the conditions for yielding in a two-dimensional state of stress the stress value obtained agrees with the yield stress of the web material obtained from normal tensile tests. The stresses indicated on Fig. 15 must be considered as comparison stresses only.

Load-web deflection curves are shown in Fig. 16 during the crippling run. The deflection curve for



LOAD - WEB DEFLN CURVES DURING CRIPPLING RUN

Fig. 16

bending panel WP7 shows that even under an applied load of 180 tons, the web deflections were very small. The reversal of direction of web deflection in panels WP1, 2 and 9 was due to change in configuration of the deflection contours. Plate 1 shows panels WP1, 2 and 3 after crippling and Plate 2 shows "Luder Lines" developed in panels WP9, 10 and 11, during the crippling run due to the presence of a layer of "mill scale" on the girder. The lines in a longitudinal direction on the girder can be seen to have concentrated towards the bolt holes for the intermediate stiffeners. Orthogonal lines were just discernible in some regions with the light shining across them, but they cannot be seen in the photograph. The more pronounced effect of the longitudinal lines may be due to the fact that they occur in the direction of rolling of the web plate.

Strain gauge readings on intermediate stiffener S3 were taken during the crippling run. From these readings values of the axial load in the stiffener during the collapse of panel WP3 were obtained. Table 6 shows the load and corresponding mean stress values for increasing girder load.

TABLE 6

Load, tons ...	150	160	165	170	175	180
Load in Stiffener tons	−1.44	−2.07	+2.57	+1.04	−3.03	−12.55
Stress in Stiffener tons/sq. in. ...	−0.50	−0.72	+0.89	+0.36	−1.05	−4.35

It is evident from the table that up to 160 tons girder load the compressive stress was extremely small. The mean stress then became tension due to some local change in the shape of the buckles in adjacent web panels, but further increase of load again induced compressive stresses which increased rapidly above 175 tons girder load. The mean stress, however, was still small at 180 tons, beyond which it was not possible to take strain readings.

After a girder load of 180 tons had been reached, loading was slowly continued until the girder reached its ultimate carrying capacity of 199 tons. Plate 3 shows the crippled girder after removal from the testing machine. The large permanent girder deflection and buckles in the web are clearly visible together with the bending of the intermediate stiffeners.

Torsion Test

Torsion tests were carried out on the girder with each set of intermediate stiffeners attached. The values of the torsion constants in each case, together with the value obtained by relaxation methods, are shown in Table 7.

TABLE 7

Intermediate Stiffeners No.	Angle Section	Torsion Constant J ins.[4]
(1)	$2/4\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$	10.93
(2)	$2/4'' \times 3'' \times \frac{3}{8}''$	10.76
(3)	$2/4'' \times 3'' \times {}^{5}/_{16}''$	9.66
(4)	$2/3'' \times 3'' \times {}^{5}/_{16}''$	9.48
(5)	$2/3'' \times 3'' \times \frac{1}{4}''$	9.51
Torsion constant by Relaxation Method ...		9.14

It was found that the torsion constant gradually reduced with reduction in the rigidity of the intermediate stiffeners, all values being slightly higher than the value of the constant obtained by relaxation methods. The difference between the mean experimental value and the theoretical value was approximately 10 per cent.

Discussion of Test Results

(a) FLANGE STRESSES

Although variation of bending stress across the girder flanges was considerable, comparison of the mean

Plate 1

experimental flange stress with the theoretical bending stress was quite satisfactory. The location of the flange gauges were fixed as far away as possible from the rivets to avoid stress concentrations, but in spite

of this stress "peaks" were indicated. Since these "peak" values did not coincide with rivet positions, and since the shape of the distribution diagrams was similar for each series of tests, it has been inferred that they were due to loading eccentricities caused by inherent lack of symmetry in the girder.

(b) Web Lateral Deflections

(1) Shape of deflection contours :—

In spite of the fact that idealised conditions were not fulfilled in the test, good agreement between the shape of the theoretical web deflection contours and those

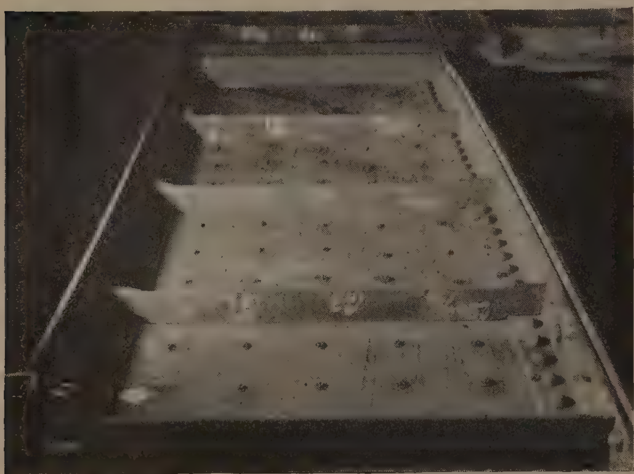


Plate 2

obtained experimentally was obtained, particularly for the shear panels ; the shapes of the buckles in the bending panels were more irregular. It follows, therefore, that the comparatively large initial web deflections have had little or no effect on the shape of the web deflection contours under load. Deflections in the majority of shear panels, irrespective of side ratio, comprised one large half-wave across the compression diagonal of the panel and two smaller half-waves in the reverse direction upward or downward, situated in the corners terminating the compression diagonal. As the maximum load was approached and the tension diagonal came into operation the main buckles were elongated and oriented if necessary to lie exactly along the tension diagonal.

(2) Load-Lateral Deflection Curves

Under ideal conditions the application of load produces no web deflections until the critical stress is reached, when deflections will be rapidly produced, giving rise to an "elastic stability phenomenon." Thereafter, the increase of web deflection is slowed down by the action of induced membrane stresses arising from the "anchoring effect" at the plate edges which induce tension stresses in the plate.

The load-lateral deflection curves obtained showed no elastic stability phenomenon. Since all the panels had fairly large initial deflections, deflections due to load were produced on first application of loading and increased continuously with the applied load until the web material yielded. Therafter, the deflections increased more rapidly, the area of yielding spreading until collapse of the panel occurred. A comparatively rapid change of web deflection was produced in panels WP1 and 2 (Fig. 16), the "kinks" in the curves being brought about by a sudden change in the shape of the web-deflection contours. Thus the occurrence of sudden collapse in practice due to elastic stability phenomena in plates is precluded to a large extent by imperfections in the plates obtainable for fabricating girders. Should eccentricities of loading exactly balance the initial out-of-straightness of the web plate a type of elastic stability phenomenon will occur, but this is not likely to happen very frequently.

Deflections of shear panels were generally larger than deflections of bending panels at loads equal to the same proportion of the Timoshenko Buckling Load and for comparable values of initial deflections. Large initial deflections, however, usually caused larger deflections under load of otherwise similar web panels. The load-web deflection curves for all the web panels did not give a rectangular hyperbolic shape and consequently use of Southwell's Method for estimating panel buckling loads was impossible.

Web Stresses

(1) "Middle Surface" Stresses

Agreement between mean or "middle surface" experimental stresses and theoretical stresses calculated on the assumption of linear distribution of bending stress and theoretical shear stress was generally quite satisfactory. In no case was the formation of membrane stresses very obvious. Some of the results showed the experimental tensile principal stresses to be larger and the compressive principal stresses to be smaller than the theoretical values ; this may have resulted from the formation of membrane stresses but the effect was relatively unimportant. Experimental shear stresses compared favourably with theoretical values.

(2) Bending Stresses

Bending stresses, caused almost entirely by web deflections, have been very high throughout the tests,

Plate 3

reaching on occasion five times the middle surface stresses at the same point. Maximum values of bending stress occurred at the crests or troughs of buckling waves and high values also occurred at the edges, thereby justifying the assumption of partly restrained edge conditions when considering web stability. Evidence was also given that increased rigidity of edge supports generally reduced the bending stress, although the difference was not large.

Although the ratio of plate bending stress to middle surface stress was high, the test results showed that at loads corresponding to the theoretical buckling loads for

web panels with simply supported edges, no sign of even surface yielding of the web plate material was apparent.

Bearing Stiffeners

The irregular distribution of stress across the bearing stiffeners was due to the practical difficulties involved in all built-up work. The summation of the axial load in the stiffener section and that in the adjacent web plate corresponded to the end reaction or concentrated load, and transmission of load from stiffener to web plate was almost uniform throughout the depth of the girder. Less than half of the concentrated load or end reaction found its way into the stiffener angles, due no doubt to the narrow bearing plates. However, it seems that the utilisation of stiffener angles and a proportion of web plate as specified in B.S.S. 449 (1949) errs on the conservative side when used in design calculations. It is difficult in view of the test conditions to give any quantitative conclusions applicable to general design, but subject to confirmation by further tests, a reduction of concentrated loads of say 25 per cent. in the design of load-bearing stiffeners seems justifiable.

Intermediate Stiffeners

Axial stresses in all the intermediate stiffeners tested were very small at loads exceeding the theoretical buckling loads for the web panels bounded by them. Some tension in the stiffeners indicated bending, but at working loads the bending stresses were very small.

Crippling Run

Buckling of a critical web panel within the elastic limit was the primary cause of failure of the test girder. Table 8 has been constructed to show the theoretical critical stresses and critical loads, B.S.S. 449 design stresses and design loads, together with the experimental elastic behaviour limit and ultimate load carrying capacity.

Agreement between the theoretical crippling loads and the B.S.S. 449 design loads estimated on the assumption of uniform shear distribution down the web is quite close. Thus the present design method allows for a factor of safety on the Elastic Behaviour limit of web panels between 1.2 and 1.4. The factor of safety on the ultimate carrying capacity of the girder or of the particular panel itself is 1.7.

Loads in the intermediate stiffeners although increasing rapidly when yielding of adjacent web panels occurred, were not large even during the crippling run. A design procedure involving the bending and torsional resistance of the stiffener section could probably effect economy in material and the desirability of further tests of limited scope along this line is indicated.

Torsion Test

The reduction in value of the experimental torsion constant with decrease in rigidity of the intermediate stiffeners towards the value obtained by "relaxation methods" is probably caused to some extent by the increased rigidity imparted to the web by the bolted legs of the stiffeners. These considerably increase the effective web thickness by varying amounts for short lengths which add up to more than 15 per cent. of the total girder length.

Comparison of Riveted and Welded Girders

As far as flange section is concerned there is little to choose between riveted construction and welded girders using flitch plates in the web. In both girders tested, the stress variations across the flange plates were very similar and were due to small eccentricities of loading. Although comparison of the web-lateral deflection curves is difficult, the results show that for web panels loaded below their elastic behaviour limit to the same proportion of their Timoshenko buckling loads the ratio of web deflection to the length of the longer panel side for the riveted girder is approximately 15 per cent. greater than that for the welded girder.

In view of the fact that the welded girder was tested under conditions different from those for which it was originally designed the model girder web was considerably over-stiffened. This is borne out by the high loads obtained in the riveted girder using a lighter web plate.

TABLE 8

Girder	Panel	Size	Theoretical critical stress		Theoretical critical load	B.S.S. 449 Design Shear Stress
G3	WP 3	$21\frac{1}{2}'' \times 30''$	7.27	9.06	115	7.35

Girder	Panel	Size	B.S.S. 449 Design Load	Elastic Behaviour Limit	Ratio of Elastic Behaviour Limit of Theoretical Critical Load	Ultimate Load	Ratio of Ultimate Load to Theoretical Critical Load
G3	WP 3	$21\frac{1}{2}'' \times 30''$	116	160	1.4	199	1.73
G3	WP 8	$18\frac{3}{8}'' \times 30''$	148	160	1.2	—	—

Acknowledgements

The authors wish to express their sincere gratitude to Messrs. Dorman Long & Co., Ltd., Middlesbrough, and particularly to Mr. S. Barlow, M.I.Struct.E., for the facilities provided to carry out the above investigation. Acknowledgement is also due to Professor R. H. Evans, of the University of Leeds, for his continual assistance.

Bibliography

[1]Mackey, S., and Brotton, D. An Investigation of the Stress Distribution in a Welded Plate Girder for the Margam Plant—THE STRUCTURAL ENGINEER, Vol. XXVIII, No. 2 (1950).

[2]Fisher Cassie, W., and Dobie, W. B. The Torsional Stiffness of Structural Sections—THE STRUCTURAL ENGINEER, Vol. XXVI, No. 3 (1948).

Some New Developments in Prestressed Concrete

Discussion on Dr. P. W. Abeles' Paper*

Errata

The following amendments should be made in the paper as printed :—

p. 259, col. 1, line 48. For " two " *read* " twelve."

p. 268, heading to Table II. For " Fig. 13 " *read* " Fig. 9."

p. 268, col. 1, line 21. For the reference number "7" *read* "9."

p. 277, reference 12 *should read* " Prestressed Concrete applied to the Construction of Railway Bridges and other Works," Railway Paper No. 44.'

On a motion from the Chair, a hearty vote of thanks was accorded the author for all the work he had done in preparing the paper, for having recorded his experiments and his results to the benefit of the profession, and for the interesting manner in which he had presented the paper.

Dr. ABELES briefly expressed his appreciation.

Mr. H. KAYLOR (Associate-Member), asked for a little more information concerning the manner in which the fatigue tests were carried out at Liège, for he was interested to know the equivalent live load to give the variation of stress from 100 or 200 lb. per sq. in. in compression and 700 or 800 lb. per sq. in. in tension.

With regard to composite construction, utilising the minimum prestress element, Mr. Kaylor asked for information as to the limiting size of the members from the point of view of economy. If we had a member without any propping, he said, it had to carry itself and the dead load of the superstructure. What about the downward deflection which might take place due to that ? Was it abnormal in Dr. Abeles' experience, or how was it catered for ? Was there a big upward camber to guard against it ?

Mr. R. H. SQUIRE (Member), pointing to a statement by the author, on page 260, that partial prestressing was much too complex a problem to be solved on an entirely fundamental basis, said he did not agree. In regard to Slab S1, he said he had made some check calculations ; to some extent they must be rough, because he had had to guess information where it was not fully given. The concrete in slabs S1, S2 and S3 was of the same composition as that in the beams, and in his calculations Mr. Squire had taken 7,000 lb. per sq. in. as the cube strength.

Calculating the failure load, Mr. Squire said he had arrived at a load on the jack of 28.6 tons, after allowing for the dead weight of the slab. On the basis of 8,000 lb. per sq. in. cube strength, the figure would have been nearly 32 tons, which agreed fairly well with the figures given in the paper.

It was interesting perhaps to note that the calculation showed that the steel stress at failure on the lowest wire would be about 95 tons per sq. in., and for steel at 100 tons per sq. in. one would certainly expect a concrete failure, which he believed had occurred. It was also interesting that the un-tensioned wires at failure would be taking a stress of about 170,000 lb. per sq. in., and that the percentage of the total tension taken on the un-tensioned wires at failure was about 30. So that they were doing quite a lot of work.

Asking for explanations on one or two matters, Mr. Squire said the dead weight of the construction given in Figs. 2 and 5 was 2.6 tons. The actual dead weight of the slab, he believed, worked out at about 4.34 tons. That was probably an adjustment, and he believed he could guess what it was, but he would leave it to Dr. Abeles to explain the discrepancy.

At the bottom of page 264, in a reference to Slab S3, it was stated that for a maximum mean tensile stress of 100 tons per sq. in., and a maximum concrete stress of 4,300 lb. per sq. in. obtained from prisms, the calculated failure load amounted to 31.3 tons. On page 265, in a reference to Slab S4, it was stated that the failure load calculated for a mean steel strength of 100 tons per sq. in. and a concrete stress of 3,000 lb. per sq. in. amounted to 31.8 tons. Those figures seemed to him to be a little inconsistent. In both cases the same steel, of the same strength, was used, and the same quantity of concrete was used, but in the second case the concrete stress was lower, and Mr. Squire estimated that the failure load in the second case would be much lower than in the first. Possibly, he suggested, there was a misprint.

Mr. A. GOLDSTEIN (Graduate), who added his congratulations to Dr. Abeles on his paper, said that it seemed that in the paper a case had been made for partial prestressing, but Dr. Abeles had not given any idea as to the technical or economic limits of his proposals.

Referring to economic limitations, Mr. Goldstein pointed to the difference between this country and the Continent in the balance between labour and material costs. On the Continent, the indications were that materials were expensive, labour relatively cheap. In this country, whilst nowadays everything was very expensive, the balance was somewhat the other way. The overall economics of the methods proposed by the author should therefore be investigated since a small saving in materials, be they steel or concrete, did not necessarily mean an economy when considered in conjunction with the labour required in the design, manufacture and erection of the proposed units.

In connection with technical limitations, he invited Dr. Abeles' views on the possibilities of using partial prestressing technique for post-tensioned construction.

Considering, for example, a large span beam bridge, it would appear that design tension stresses of 600-800 per sq. inch would be permitted. However, in post-tensioned work crack control was exceedingly difficult ; a shrinkage crack might well develop at or near a

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Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Wednesday, October 24th, 1951, at 6 p.m., Mr. Walter C. Andrews, O.B.E., M.I.C.E. (President), in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXIX, No. 10, pp. 259-278. (October, 1951.)

critical section and this could have two results. If the tension face was the soffit, large cracks might occur which would not close and would produce higher compressive stresses at the remote surface. If the tension face was the upper surface under no super load then a neutral axis shift and consequent increase of compressive stresses could well lead to instability and subsequent failure.

He therefore doubted the wisdom of using those high tensile stresses for design loads in post-tensioned construction, and felt that Dr. Abeles' views on the limitations of his proposals would be of great interest.

Mr. M. B. MACRAE illustrated some of the points mentioned in the paper by means of slides.

Slide 1 showed a precast concrete parapet of the kind used during early stages of the type of bridges mentioned in the paper. It was not altogether an ideal solution, and, in addition, the Ministry of Town and Country Planning asked that the Railway Executive should reproduce the previously existing types of parapet as far as possible. Thus, following Dr. Abeles' line of using a minimum amount of prestressed concrete, they had designed the "U" type of parapet beam.

Slide 2 illustrated the "U" type of beam. The "box" was factory-made and filled with *in situ* concrete. This beam was comparatively cheap and capable of carrying a brick or masonry parapet. It was not necessary after erection to provide any shuttering on the outside of the beam.

Slide 3 illustrated one of the completed bridges at Gorton.

Slide 4 illustrated a possible development, applied to a water tank, which was being considered, involving high tensile wires, carried in grooves and subsequently grouted. He pointed out that the floor shown on the slide was cantilevered out from the main supports, and under conditions of full loading there would be no tensile stresses and no chance of cracking, which was a most desirable condition for water tanks.

Slide 5 showed a platform awning in prestressed concrete, a scheme which at the moment was in the draft stage. The cables and wires were carried round at one side, both ends being anchored at the other, it being possible to tension from one side only, on account of railway traffic. There was also shown an outline sketch of a platform roof, with columns at 65 ft. centres, the roof being a thin slab, carried on a beam about 3 ft. max. depth, which was situated above the slab, thus providing a flat soffit to prevent shadows, when artificial lighting was in use.

Mr. H. T. HORSFIELD illustrated the design of a partially prestressed roof beam which was prepared recently in the Civil Engineer's office of the Eastern Region of British Railways.

The beam was required to span a distance of 64 ft. 7 in., across a running shed, with a cantilever extending 37 ft. 6 in. over a repair bay, there being no support whatsoever to the beam at the repair bay end.

The beam was of "I" section. The ultimate resistance was calculated on the plastic stress distribution to withstand the live bending moment with a load factor of 2.5, and the dead load moment with a load factor of 1.5. Thus, whatever cracks might or might not occur, the beam would carry the load required and would have a large factor of safety. As the compression flange was of constant cross-section, the depth was made approximately proportional to the ultimate bending moment.

The working load caused tensile bending stresses in the upper flange over one support and in the lower flange at midspan in the running shed. Prestress was provided to limit the actual tensile stresses occurring in the concrete to what was considered a reasonable amount, so that no cracking would occur under working load. It would have been possible to have provided one cable in the top flange and a separate one in the bottom, with four anchorages in all; but it was considered more economical to have the cable made continuous, being raised across the web from the bottom to the top flange. In view of the uncertainty regarding friction losses in cables at bends, the cable was kept as straight as practicable. As the dead weight of the beam would counteract part of the prestress as it was applied, a high prestressing force could be used without overstressing the concrete. A higher effective prestress meant, of course, that a smaller concrete cross-section was satisfactory.

The beams were designed to be cast on the ground at the site and to be lifted at the two support points. Consequently, no special handling stresses had to be considered. The live working load represented snow, or men working on the roof. The full value of 20 lb. per sq. ft. over the whole roof would occur only on very few occasions, and a tensile working stress of 650 lb. per sq. in. could safely be allowed in the concrete; in that condition, no cracks should occur.

Referring to the untensioned wires, Mr. Horsfield said that when the tensile strength of the concrete was utilised in design, in partial prestressing, it was often found that only part of the wires, needed for the ultimate strength, had to be tensioned. In that particular case only three-quarters were to be tensioned. There was no need to make the untensioned wires continuous, and they were designed as two separate groups. They had no special anchorages, but were to be set in position and concreted in.

In the normal simply supported beams, he continued, the maximum shear stresses occurred in the zones of minimum bending moment, and *vice versa*. In that particular roof beam, however, the maximum shear force and bending moment occurred at the same point. Thus, the principal tensile stresses at that section were investigated under ultimate loading conditions, with assumed distributions of bending and shear stresses. As prestressed concrete design was applied to continuous beams, it became increasingly urgent to investigate experimentally the effect of combined shear and bending stresses on ultimate strength.

Finally, dealing with the shape of the beam, Mr. Horsfield said the upper surface was pitched at 4 deg. to suit the roof sheeting employed. The ends of the repair bay were closed by doors, and that section of the beam had to have a level soffit. The running shed had open ends, so that a curved soffit could be used, giving a suitably varying depth to the beam, as required for an economic design. A continuous brick wall between the repair bay and the running shed meant that from inside one would see either the flat soffits or the curved soffits. The appearance of the beam was said to be unconventional; but he asked what should a conventional prestressed beam look like.

Dr. ABELES, replying to the discussion, said he was grateful to those who had criticised him, because without that criticism he would not have been able to progress. Criticism was most valuable; either one was wrong, then the question had to be reconsidered and adjusted according to requirement; or it was possible to give an ocular proof that one was correct.

Replying to Mr. Kaylor, he said the limits of the bottom stress at Liège of +100 and —550 lb. per sq. in. corresponded to those on the bridges under dead and live load respectively. Under dead load there was only

compression ; and it was always important to ensure this compression, so that any crack which might have developed would close up. After the first million repetitions of a loading corresponding to working load conditions carried out within a few days at four repetitions per second, the cracks closed up entirely on removal of the load, which was a most amazing feature.

The second and third million repetitions were at much higher loading, the highest loading being imposed during the third million repetitions. If, after the third million repetitions, the static loading had been 66 per cent. of an ordinary static failure load, he would have been very pleased. But the tests showed that it was approximately 100 per cent. That gave an indication of the great success of the tests.

The second question by Mr. Kaylor, concerning the limiting size of the members of the combined sections, was very difficult to answer. If the structure were designed as he recommended, with no tensile stress occurring under dead weight, it must be prestressed to such an extent that under ordinary load conditions the whole bottom flange was in compression. The deflections (be it the upward camber before the additional concrete was placed or the downward deflection under load) were very small. Obviously, if one had a solution using props there were different questions to be investigated ; but the time available was too short for him to attempt to discuss them at the moment.

Referring to Mr. Squire's comments, he said the word "complex" in the sentence quoted from the paper in regard to partial prestressing did not relate to ultimate load conditions for which he had based his calculation on certain basic principles ; but he meant that when cracks had occurred in a system with tensioned and untensioned wires, the cracks closed up on a reduction of a load. The word "complex" related to the state of stress in a cracked section before the cracks closed. As Mr. Goldstein had mentioned, when a crack occurred, there ought to be a shift in the neutral axis, but apparently this did not influence the later behaviour. This question could be clarified only by tests. With regard to the ultimate load condition he referred to Fig. 2 in his paper. On the right part of the figure it was shown how the ultimate load was balanced. The bending moment for the self weight of approximately five tons corresponded to a smaller point load, as shown in the figure.

Dealing with Mr. Goldstein's reference to cracks in very large bridge structures before they were prestressed, he said Professor Magnel had mentioned the occurrence of such shrinking and settlement cracks. Obviously, if such cracks occurred, particularly on the upper side of a beam, it was a matter which must be well considered, and Mr. Goldstein's remarks were quite important. In the special design which Mr. Horsfield had illustrated, the manufacture of the beams should be carried out in such a way that no settlement occurred such as might happen in very long bridges. Settlement in a bridge construction could be avoided or could be made less likely to occur if non-tensioned reinforcement was provided. In any case, the tests described had shown that, even if there were cracks, and even if they occurred a million times, they would be harmless. (The time being too short to allow him to reply to this point in more detail and to the question of economy, Dr. Abeles said he would deal with it in a written reply.)

Written Discussion

Professor A. L. L. Baker (Member of Council) writes : Dr. Abeles should be commended for his great persistence in developing partial prestressing, in spite of much criticism and disparagement. The significance of the process, it would appear, can be clearly appreciated from a super position of the stress-strain curves of the prestressed and non-prestressed steel. I should be pleased if Dr. Abeles would be kind enough to comment on this.

Fig. 1 shows OAB the stress-strain curve of the prestressed high tensile steel in a beam, and O′CD the non-prestressed high tensile steel or O′CE, the non-prestressed mild steel. OO′ represents the pre-strain in the steel

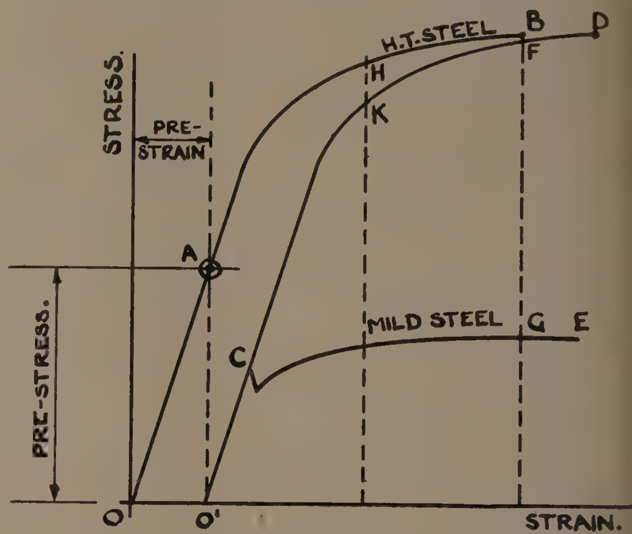


Fig. 1

and O′A the prestress, so that to the right of A at any stage of loading the strain in the non-prestressed steel has a value denoted by the intersection of the vertical ordinate passing through the corresponding strain value of the prestressed steel. Thus, when the prestressed steel develops ultimate strain value at B, the stress and strain in the non-prestressed high tensile steel is represented by the co-ordinates of F referred to the origin O′, or in the case of mild steel by the point G. It is seen that it is only necessary for the steel to extend plastically a small amount before failure in order that the non-prestressed steel may develop a stress very close to the ultimate value. Thus, the non-prestressed steel before failure can very nearly develop its full strength. If it is required to restrict the strain to a value denoted by such points as H and K in order to provide sufficient depth of compression zone in a beam, the full strength of the non-prestressed steel can still be very nearly developed. If a proportion of the steel representing about half the available total ultimate strength is prestressed, the pre-compression force may be halved as compared with full prestressing, thus allowing a reduction in the size of the bottom flange. A factor of safety against cracking is provided which is sufficient for many purposes since cracking will not occur until a compression force about equal to half the ultimate total steel strength is developed by bending. The strain required to develop full steel strength is, however, greater than that required when full prestressing is used, but in most cases this will not excessively reduce the total ultimate compression strength available in the concrete by raising the position of the neutral axis. This possibility should be checked when it is not quite obvious that adequate ultimate total concrete compressive strength is available.

The pre-compression force can also be reduced by using a smaller value of prestress in the whole of the reinforcement. Partial prestressing has the advantage

sing part of the steel to oppose the pre-tension before y load is applied.

Mr. R. H. SQUIRE writes : The ultimate load figure en in the discussion took no account of the four ⅜ in. m. M.S. bars. Corrected figures are as follows : For S2, and S3, with steel at 100 tons/sq. inch and ncrete cube strength of 7,000 lb./sq. inch, M_{ult} = 80,000 in. lb. and maximum deflection 6.35 in. For with the same steel strength and a concrete cube ength of 5,000 lb. per sq. inch. M_{ult} = 3,155,000 in. lb. d maximum deflection 4.73 in. In both cases concrete lure would be expected : the accuracy of the results pends of course on the accuracy of the data used.

The corresponding figures using rectangular distribu- n and steel at 100 tons/sq. inch are for S1, S2 and S3 th concrete at 4,300 lb./sq. in. M_{ult} = 3,860,000 in. lb., d for S4 with concrete at 3,000 lb./sq. inch 3,620,000 lb.

As the loading arrangement for S1 and S2 differs from at for S3 and S4, the jack loads are not directly mparable. The actual values for the ultimate oments reached appear to be :—

S1—3,693,500 in. lb.
S2—3,755,000 ,,
S3—3,863,000 ,,
S4—3,428,000 ,,

The value reached in the case of S4 seems surprisingly gh in view of the concrete mix used for the added ncrete, and the fact that the compression zone is ainly of this quality.

The fact that Slab S2, with the two most efficiently aced wires already fractured, reached a slightly higher ad than S1, seems to confirm that the concrete would il first in all cases, especially as the added concrete this case showed the highest prism strength.

The author's paper deals extensively with partially estressed construction, and it is certainly true that gh strength steel not prestressed, used as in these abs, will have a high degree of stress at failure ; but should be realised that both on account of the lack of estress and because of its position nearer to the utral axis, it cannot in the example considered reach e same stress as that in the prestressed steel lower own, even at failure.

The author's figure of 101 per cent. of the ultimate rength on 58 wires at the top of page 266, is based on rectangular " stress distribution ; this method is only rough approximation and the result here is mis- ading. Another instance where this system is mis- ading is the author's application to partially pre- ressed construction, as the same result (calculated) ould be obtained whether 10 wires or 60 were pre- ressed, which fairly obviously is incorrect.

In the writer's opinion a fundamental approach is ways preferable, and particularly so in this case of artial prestressing, as it gives a fairly accurate under- anding of conditions at all stages of loading, as well as me guidance on the probable deflection : for research ork it would seem to be essential.

It is interesting to note that the German Draft Design pecification for Structural Members in Prestressed oncrete, published in 1950, does in fact propose a osely fundamental treatment.

Written Reply by the Author

Mr. Goldstein has put forward the question of economy nd mentioned that the cost of labour was of greater fluence than that of material. This may be so with ctory-made precast concrete, as it is the case with pre- nsioning. The price, ex works, depends mainly on the concrete quantity, since in most cases depreciation of plant, overhead and profit are charged on the concrete quantity sold.

With regard to post-tensioning, the cost of tensioned cables with two anchorages (including the provision of ducts, the supply, placing, tensioning and grouting of cables) may generally be taken as $a + L(b + c)$ where "L" is the length of the cable, "a" a fixed sum and "b" and "c" the respective costs, per ft. run for steel and for labour, including grouting. For example, for 36 post-tensioned wires 0.2 in. dia. in Freyssinet or Magnel-Blaton cables the value "a" may be taken as an amount between £10 10s. and £13 10s. "b" as 2s. 11d. (for steel at 80s. per cwt.), and "c" as between 3s. 7d. and 4s. 6d. per ft. If $L = 100$ ft. the entire cost would amount to between £43 and £50 11s. 8d. Assuming that with partial prestressing the tensioned wires are reduced to 24, then the cost of tensioned wires is obviously reduced to two-thirds, and a saving of approximately 20 per cent. or more is possible, when a third of wire is placed non-tensioned in the concrete at casting over a length of, say, $2/3\ L$.

With regard to Mr. Goldstein's reference to cracking of the concrete before prestressing, the author would like to amplify his first reply by saying that in order to avoid shrinkage cracks in the tensile zone it would be possible to provide in very long beams thin separation joints, representing artificial cracks where the bending moment is smaller and where tensile stresses do not occur under working load. Another means would be to provide a facing of concrete under higher stresses by casting and prestressing in two stages, as shown by the author in the discussion to paper[12].

The author is greatly obliged to Professor A. L. L. Baker for his support with regard to partial prestressing, clearly stating its advantages. Professor Baker's Fig. 1 is in close agreement with the author's Fig. 13 in paper[10].

Mr. Squire emphasised twice the importance of a "fundamental" approach, implying that the author's approach was not fundamental. As the author mentioned in the paper, his approach was not entirely fundamental, which would be a matter for a highly-trained physicist, capable of using new research on the internal structure of matter. A fundamental approach depends on determining the true roots or basis. When these roots are known a fundamental approach would commence from them. However, in a state of development the true roots are often not yet known and it is necessary to make certain assumptions. The validity of the fundamental approaches thus depends on the validity of the assumed bases. Unfortunately, the basic stress distribution in a cracked section with well-bonded steel reinforcement has not yet been experimentally established and it is necessary to make an assumption and to check its validity indirectly. Thus an approach which claims to be fundamental will become a fundamentally wrong approach, if it has been established indirectly by observations that the basic assumption must be incorrect.

Mr. Squire's criticism resembles that of Mr. Gifford, in a written contribution to Mr. King's paper[29]. Mr. Gifford believes the assumption used by the author to take for under-reinforced beams with well bonded reinforcement "a rectangular compressive stress block and steel at its ultimate strength," has been chosen only because it is the "simplest" solution and states "the fundamental error made is to assume the steel reaches its ultimate strength, which involves ignoring strains." This point may also be behind Mr. Squire's criticism of the author's method, but it is based on a wrong assumption. It is very important to clarify this point. All the strain

measurements extending across the depth of the section show up to failure an approximate straight line strain distribution over the section from which one could conclude, as Mr. Gifford did, that the ultimate strength in the steel could never be reached, since the strain is much too small. However, these strain measurements extend over several centimeters and represent the average of high strain in the crack itself and low strains where the steel is still bonded to the concrete. There are many instances where the steel fractured and consequently no doubt can exist that the ultimate strength was reached, while in other cases similar states of cracking and deformations were reached without steel fracture. In all instances, including the case of fractured steel reinforcement, the measured average strain at failure corresponds to a straight line strain distribution and is, in the plane of the tensile steel reinforcement, much below the value corresponding to the ultimate stress. It is not the average strain which matters but the strain in the crack itself. Although nobody has yet been able to measure the strain within a crack it can be concluded with certainty that in under-reinforced beams with well-bonded steel members the failure stress reached the ultimate strength or is very near this strength whatever the magnitude of the measured average strain. At the presentation of his paper[19], Dr. Hajnal-Konyi showed strain measurements which confirmed this. Although non-tensioned wires of a strength of 120 tons per sq. in. fractured, the average strain was very low and did not correspond to this stress.

The author has based his approach on two fundamental principles, i.e., (1) In under-reinforced beams with well-bonded wires the ultimate tensile resistance T_{ult} equals A_t, t_{ult}, or $A_t.t_y$, where a distinct yield point stress t_y occurs ; and (2) A straight line stress distribution of a homogeneous material can be considered as correct in a section with a closed crack. It has been observed that a crack closes completely on reduction of the load as soon as the theoretical tensile stresses reverse into compression owing to the effective prestress. The author has taken into account these two fundamental principles in the present paper, as he did in the two previous papers [1]and [10], and in the publications[9], [22] and [23]. However, he considered the question of the stresses in a section with open cracks as not yet clarified, since obviously a different stress distribution applies at that stage, before the normal straight line stress distribution is resumed on closing of the crack. Although under fatigue tests, such cracks closed after being opened a million times and are quite harmless, it would be very interesting to learn more about the stress distribution in a crack of a member with well-bonded wires. In this case the bond is destroyed only in the vicinity of the crack and the steel stress is quite different from that in ordinary concrete reinforced with steel where there is no bond over a large length of bar.

The assumed rectangular stress distribution of the concrete is an approximation which is very simple and gives satisfactory results, but has nothing to do with the principle of approach considered in condition (1). In fact this condition was included in the " First Report on Prestressed Concrete " in clause 7 (b), page 11, where the Committee agreed on the following statement : " However, it is possible to calculate quite accurately the ultimate strength of under-reinforced beams with bonded wires as the tensile force in the steel at failure multiplied by the lever arm. With pre-tensioned wires, the maximum steel stress can normally be assumed as the ultimate tensile strength..." And further, it is stated: " The lever arm may be computed according to any known method, there being but little difference in the result." It has been recognised that the question of the approximation of the concrete stress distribution does not appreciably influence the result.

Mr. Squire states that the author's analysis with the same ultimate stress for the tensioned and untensioned wires is " fairly obviously incorrect " and " misleading " ; he favourably compares the German Draft Design Specification 1950, proposing " a closely fundamental treatment " with the author's investigations.

Firstly, with regard to the German Specification 1950, it may be stated that this relates to the first form of partial prestressing as suggested by Emperger[3], where untensioned bars are used as employed for ordinary reinforced concrete. According to the German draft, the tensile bending stress under working load is limited to avoid the occurrence of cracks. The untensioned steel bars are to be designed for a cracked section based on limited permissible stresses corresponding to those in ordinary reinforced concrete not exceeding 28,400 lb. per sq. in. and double this value should not be exceeded at ultimate load. The use of high strength wires for the untensioned reinforcement has been developed only in this country, and when comparing this with the use of ordinary bar reinforcement in Germany, the author cannot understand why, according to Mr. Squire, the German proposal is a "closely fundamental treatment" but not the type of investigation indicated in Fig. 13 of paper[10] and in Professor Baker's Fig. 1. In fact, Professor Baker's diagram contains both solutions, a low strength and a high strength non-tensioned steel

In particular, the following may be said in reply to Mr. Squire's remarks. It is true the four mild steel bars $\frac{3}{8}$ in. dia. have been neglected as their influence on the ultimate resistance is significant. Mr. Squire had already stated in his verbal contribution that he would certainly expect a concrete failure for the slabs tested. The author would like to refer to the " First Report on Prestressed Concrete " where, under 7 (b), pages 10 and 11, this question has been clearly defined, representing a compromise view to which all agreed. " Under-reinforced " beams are considered as such beams in which failure occurs either owing to " (1) fracture of steel," or " (2) excessive elongation of steel followed by crushing of concrete." As indication for a safe (i.e., lower) limit for under-reinforced rectangular sections, a maximum bending moment of $0.225\ b.d^2.c_u$ is given which would amount to $0.225 \times 37 \times 10.48^2 \times 6000 =$ 5,640,000 lb. in. for a cube strength of 6,000 lb. per sq. in. (corresponding to Slab S3) ; this is much more than the maximum bending moment of any slab S1—S4. It can be stated that these slabs failed primarily in the steel, while crushing of the concrete followed after excessive steel elongation.

It would appear that Mr. Squire came to the conclusion of a failure due to concrete combined with steel stresses much below the strength on his fundamental approach by assuming a definite curve for the concrete stress distribution and a high cube strength as the maximum stress, resulting in a large lever arm and a reduced steel stress. Moreover, it would appear that Mr. Squire assumes a straight line triangular stress distribution for the steel at failure, while the two diagrams, Fig. 13 of paper [10]and Fig. 1 of Professor Baker's contribution clearly show this cannot be the case.

The author is at a loss to understand Mr. Squire's statement : " The fact that slab S2 with the two most efficiently placed wires already fractured, reached a slightly higher load than S1, seems to confirm that the concrete would fail first in all cases." When viewing the load deflection diagrams of slab S2 which was shown at the lecture, one can observe that the load was no

reduced when the jack reached its maximum extension but was continued after underpacking of the jack. With the other slabs the load was reduced to zero and a 4th loading applied as seen in the Figs. 2 and 5 of the paper. By avoiding a reduction of the load and a re-loading at a stage near to failure, it was possible to reach with slab S2 a higher failure load and a much greater deflection than with the other slabs. It may be mentioned that previous to testing the slabs S1 and S2, it was ascertained that the steel strength of the wires used in these slabs was less than the required strength specified, and the assumed value of 95 tons per sq. in. is rather on the high side. The ultimate tensile force of 58 wires amounted for this strength to 387,600 lb.

The concrete strength obviously influences the lower lever arm of the internal forces. But for a definite concrete strength there is "but little difference in the results," "for various known methods," as stated in the "Report," page 11, mentioned before, and thus there should be little variation from the computed value of 9.63 in. for the prism strength of 6,150 lb. per sq. in. The theoretical bending moment at failure is thus $387,000 \times 9.63 = 3,733,000$ lb. in., while the actual bending moment amounts to $\frac{1}{2}(35.5 - 2.7) \times 2,240 \times 102 = 3,747,000$ lb. in., where 35.5 Engl. tons corresponds to 36.1 m. tons. This is 0.4 per cent. (not as stated in the paper, 1 per cent.) in excess of the theoretical value. The four mild steel bars $\frac{3}{8}$ in. dia. with a lever arm of 5.5 in. between the two compressive and two tensile bars, are able to take up a maximum bending moment of $2 \times 0.11 \times 35,000 \times 5.5 = 42,000$ lb. in., assuming a yield point stress of 35,000 lb. per sq. in. The maximum calculated bending moment thus amounts to 3,775,000 lb. in. which is 0.8 per cent. above the actual bending moment ; thus it can be stated that the two bending moments are approximately equal. The author would like to draw Mr. Squire's attention to the Figs. 2 and 5 referring to slabs S3 and S1 respectively. In slab S1 the concrete prism strength was 5,950 lb. per sq. in., and the steel strength 95 tons per sq. in., and the actual failure moment amounted to $\frac{1}{2}(34.9 - 2.7) \times 2,240 \times 102 = 3,678,000$ lb. in., while in slab S3 with the lower concrete prism strength of 4,300 lb. per sq. in. but a higher steel strength of 100 tons per sq. in. the actual failure moment was $\frac{1}{2}(30.8 - 2.6)\ 2240 \times 102 = 3,826,000$ lb. in. These results are quite in agreement with the strength properties of the steel, but do not comply with Mr. Squire's assumption that the concrete strength directly influences the failure resistance.

These test results, particularly with regard to S2, are further confirmation of the author's basic condition (1), according to which the ultimate steel strength is reached, or approximated, in under-reinforced beams with well-bonded wires independently of whether all wires are tensioned or only a part. In this respect reference may be made to the author's contribution[30] to the discussion of paper[19] that even non-tensioned high strength wires fracture at failure. Further reference may be made to the tests at the Brixton School of Building[1] and [31].

Mr. Squire states that the assumed rectangular stress distribution is a rough approximation. It is certainly an approximation but the results obtained show a very satisfactory agreement with actual values, as the author has ascertained when investigating many test results of prestressed and ordinary reinforced concrete. It is evident that a stress distribution of a definite shape with a maximum stress equalling the cube strength cannot comply with concretes of different properties. In a concrete of very high strength the stress distribution at failure may approach a triangular stress distribution with a relatively high maximum stress and in a concrete of lower strength a rectangular distribution may be approached. The consequence is that any calculation based on a definite shape assuming that the cube strength is reached is bound in most cases to lead to wrong results, while the approximation based on a maximum stress which may be between 0.6 and 0.8 of the cube strength in accordance with its plasticity is more suitable.

The load referred to by Mr. Squire, of 31.8 tons, which was stated to be the calculated failure load of slab S4, was less than 33.9 tons which was given for slab S3 in Fig. 5 ; the actual corresponding loads on the jacks were 27 and 31.3 tons respectively.

It appears from Mr. Squire's contribution that the author's paper was not clear enough in some points and thus Mr. Squire came to wrong conclusions, and he hopes that the preceding explanation will clarify the various points raised.

[29] "A Fundamental Approach to Prestressed Concrete Design." Discussion on Mr. J. W. H. King's paper, THE STRUCTURAL ENGINEER, December, 1951.

[30] THE STRUCTURAL ENGINEER, January, 1952.

[31] "The Ultimate Resistance of Prestressed Concrete Beams," by P. W. Abeles, CONCRETE AND CONSTRUCTIONAL ENGINEER, October, 1951.

Book Reviews

Roads and Road Construction Year Book and Directory, 1950-51. (London : The Carriers Publishing Co., Ltd., 1951.) 433 pp., $8\frac{1}{2}$ in. × $5\frac{1}{2}$ in. 15s.

In the recent edition of this useful Year Book all the directory information has been brought up to date, and a new part has been included on suppliers of roadstone, in which the various firms concerned are listed alphabetically in county order and details of the types of stone supplied are given.

The existing sections in the part on the Review of Research have been rewritten and new sections on Traffic and Road Layout and Accident Analysis have been added.

The Practical Engineer Pocket Book, 1951. Edited by N. P. W. Moore. (London : Pitman & Sons, 1951.) 744 pp., $3\frac{1}{2}$ in. × $5\frac{1}{2}$ in. 8s. 6d.

In this sixty-third edition of the Practical Engineer Pocket Book, the type has been completely re-set and many of the illustrations re-drawn.

The book, intended mainly for mechanical engineers, contains the following twenty-two sections :—General Information ; Pipes, beams, columns, springs, etc. ; Friction and power transmission ; Cranes and lifting tackle ; Pyrometry ; Metallurgy ; Steam ; Steam generation ; the Steam Engine ; Locomotive practice ; Steam Turbines ; Air Compressors ; Air and Ventilation ; Hydraulics ; Machine Tools, Modern Lubrication ; Welding and Cutting ; Lighting for Workshops and Offices ; Unified Screw Threads.

A list of technical journals is given, including a selected number of American journals, and short technical dictionaries in German-English, French-English and Spanish-English complete the book.

Cold Formed Sections in Structural Practice with a Proposed Design Specification*

Written Discussion on Paper by W. Shearer Smith, A.M.I.C.E., A.M.I.Struct.E.

Written Discussion

Professor GEORGE WINTER, of Cornell University, New York, writes : In April, 1946, the American Iron and Steel Institute published the first American Specification for the Design of Light Gauge Steel Structural Members, which is nationally recognised in the United States as the governing code for cold formed structural sections. During the five years of its existence hund eds of millions of square feet of light gauge steel roof deck, wall and floor panels, and many thousands of tons of cold formed steel framing have been erected to this specification. The research work which resulted in this code. has been carried out continuously since 1939 at Cornell University under the writer's direction, and is still in progress. A second, greatly expanded edition of the specification is now in preparation.

For this reason the writer was greatly interested in Mr. W. Shearer Smith's paper, "Cold Formed Sections in Structural Practice with a Proposed Design Specification," in the June, 1951, issue of THE STRUCTURAL ENGINEER. He noticed with some regret that except for one rather minor point (provison 4 (c), the Glasgow investigators apparently did not make any use of the American Specification and the Cornell University research work that led to it. The detailed results of this research have been made available to the profession in numerous publications, about half of which are included in Mr. Smith's list of references, even though all except one are not referred to in the paper proper. A detailed Correlation of Cornell University Research Investigation and Specification for the Design of Light Gauge Structural Steel Members, 2nd ed., 1950, is available on request from the American Iron and Steel Institute. The Cornell investigators, in turn, would have been glad to profit from the results of their colleagues in Glasgow, had these results been accessible in available publications.

This is not the place to indicate in detail the numerous differences between these two specifications. Only two will be mentioned since they are believed to be fundamental.

Mr. Smith suggests the use of the same factor of safety for thin-walled cold formed sections as is used for hot rolled shapes in B.S. 449. In contrast, the American Specification uses a factor 12 per cent. larger than in the corresponding code for hot rolled steel construction (A.I.S.C.). Factors of safety are supposed to account for degrees of uncertainty inherent in designs. The percentages of dimensional tolerances, particularly with regard to thickness, are considerably larger in sheet and strip steel than in hot rolled sections. Material rolled within the prescribed tolerances cannot be rejected by the purchaser even though its thickness may be below the nominal value within the permissible limits. The consequent strength deficiency must therefore be reflected in lower permissible design stresses or, in other words, in a larger factor of safety. In addition, the more limited experience with this type of construction as compared with customary steel structures (at least in 1946, when the American Specifications were issued), also suggested a more cautious approach to this question.

In his suggested stresses for stiffened flat elements (Table 4, etc.), Mr. Smith takes no account of the post-buckling strength of flat, stiffened compression plates, but bases his values exclusively on the results of the small deflection theory of plate buckling. This approach, which leads to a great loss of usable strength, has been abandoned in aircraft design about two decades ago. In such design, as well as in the American specifications, this post-buckling strength is used fully by means of the effective width method. It has its main effect for higher b/t-ratios, say above 60 to 80. For instance, for $b/t = 216$, Mr. Smith's Table 4 gives a permissible stress of 0.43 tons/sq. in. The actual usable strength, maintaining Mr. Smith's safety factors, is more than five times as large. For Mr. Smith's sections, with their comparatively small b/t-ratios, this factor is of relatively small though still significant influence. However, development in the United States has shown the main field of application for cold formed sections to lie not so much in structural shapes of the more common type (channels, angles, etc.), but rather in decks and panels for floors, roofs and walls. In such members, b/t-ratios are very large, up to 500 in some cases. Had Mr. Smith's values been adopted in the American specifications this largest field of cold formed section application could not have developed at all.

It is realised that for economic and other local reasons, specifications and codes in various countries often differ considerably, even though they deal with the same engineering fields. Yet, international co-operation is often profitable. In recent years American engineers have learned a great deal about prestressed concrete from the much more advanced work of their European colleagues. With all due modesty, may one suggest that the British engineers may save some effort and gain some worthwhile information with regard to cold formed steel construction by availing themselves of accumulated practical and research experience in the United States.

Dr. A. H. CHILVER writes : The writer's comments are confined to the author's technique of strut design and some of the matters treated by the author under that general heading. So far as thin-walled struts are concerned the basis of the author's design technique is the equality of the strengths of a strut, corresponding to the various possible modes of failure. The concept of equal strengths was put forward by Moir and Kenedi in 1948[17], in relation to the design of thin-walled columns. The conclusion drawn by Moir and Kenedi was that the simultaneous collapse of a column and its plate components corresponds to the maximum strength of that column. Moreover, it was suggested by Moir and Kenedi—and in this respect the author concurs—that efficient and economic design is in general achieved only in the equality of column and plate strengths.

**The MacLachlan Lecture, 1950, given before the Institution of Structural Engineers, at 11, Upper Belgrave Street, London, S.W.1, on March 1st, 1951. Published in* THE STRUCTURAL ENGINEER, *Vol.* XXIX, *No. 6, pp. 165-178 (June, 1951), and No. 7, p. 209 (July, 1951).*

The writer's impression is that equal column and plate strength may not in general lead to the most efficient design of struts. In the few examples of strut design discussed by the author, in an Appendix to his paper, it is difficult to assess the relevant efficiency factors. Strut design is essentially the problem of finding a suitable member of a given length to support a given load. Another quite distinct problem is that of finding the maximum safe load which a given strut may support. But more frequently, as in roof trusses, the former problem is encountered. With this particular aspect in mind, it is interesting to consider a framework in which an isolated strut member is to be designed ; the strut is pinned at each end, and is to support, say, a load of three tons over a length of 60 inches. The component may be stanchion or column. What is the most efficient steel section to employ under these conditions ? Assuming that a cold-formed strut is to be used, the author would advocate a design on the basis of equal column and plate strength. Many types of sections are available ; suppose, however, that only plain channel struts are considered.

An analysis shows that there are at least three distinct plain channel sections each supporting three tons over a length of 60 inches, if design is based on equal column and plate strength. The dimensions of these sections are tabulated below. A comparison of the three sections shows complete inefficiency in two of them ; there are, of course, more solutions available in other types of single and composite sections. The example shows, however, that design on the basis of equal column and

Web in.	Flange in.	Wall Thickness in.	Area in.^{2}	Working Stress tons/in.^{2}	Area Ratios
5.38	1.86	.092	.821	3.65	1.14
3.67	1.98	.104	.771	3.93	1.07
2.23	2.10	.116	.719	4.17	1.00

plate strength may not provide a unique efficiency solution. In the absence of increased plate strength, equal column and plate strength of a section serves only to indicate the maximum length at which a strut may be used at its maximum load. This, however, is not the primary factor in efficient design, in which the concern is with the most efficient section to support a given loading under given working conditions. In this latter sense a single attempt at design on the basis of equal column and plate strength may not necessarily give the most efficient strut.

The author points out that when there is no tendency to overall collapse of a strut then the plate components appear to receive increased support along their edges, thereby increasing the values of the critical stresses. That there may be an increased plate component strength is of great practical importance since it may be possible to increase the working stresses of certain sections, and consequently bring about their more efficient use. In Fig. 4, the author has indicated the increased permissible stresses for the plate components when the slenderness ratios are not greater than about 85 and when there is no tendency to overall collapse. It is not clear from the author's paper whether the information may be applied also to stiffened plates by a suitable conversion of the b/t ratios. As an example of the way in which the unstiffened plate component stresses may be increased it is interesting to consider the compression of a single plain channel strut, having the dimensions $6 \times 3 \times 0.116$ in. When the slenderness ratio is greater than 128, column failure occurs ; when the slenderness ratio is equal to 128, equal column and plate strength gives a safe load of about 3.5 tons. When the slenderness ratio is less than 128, failure is predominantly that of the unstiffened flange. If the slenderness ratio is not greater than 85, then from Fig. 4 the safe load is about 5.7 tons, which is based on the increased plate component strength. At this latter slenderness ratio of 85, the working load is increased by more than 60 per cent. Behaviour of the section between slenderness ratios of 85 and 128 is not clearly indicated in the author's paper. It appears, however, that when equal column and plate strength gives a safe working stress below about 2.7 tons per sq. in., then a more efficient slenderness ratio may be found for that strut. Under these conditions equal column and plate strength fails to give maximum load carrying capacity. It is interesting to compare a design on the basis of increased plate strength with a design on the basis of equal column and plate strength ; to replace a $6 \times 3 \times 0.116$ in. plain channel section supporting a load of about 5.7 tons at a slenderness ratio of 85, the only reasonable plain channel section, designed on the basis of equal column and plate strength and working over the same length, has the dimensions $7.13 \times 2.54 \times 0.128$ in. This latter strut supports a load of about 5.7 tons over the same length as the previous strut, but it has a greater area, a lower safe working stress and is less efficient.

The author gives only the increased plate strengths for slenderness ratios not greater than 85. If more information were available it would be apparent that when there is no tendency to column failure there is always some increased plate strength, the actual increase depending upon the remoteness of the column failure.

In conclusion, the writer has compared the expression in the author's Fig. 4 with the exact theory of local instability as put forward by Lundquist. In the total absence of column instability it is found that the elastic local buckling stress of a section may be represented by an expression (in the author's form), of the type

$$f_c = \frac{C_2 E}{1 - \sigma^2}\left(\frac{t}{B}\right)^2$$

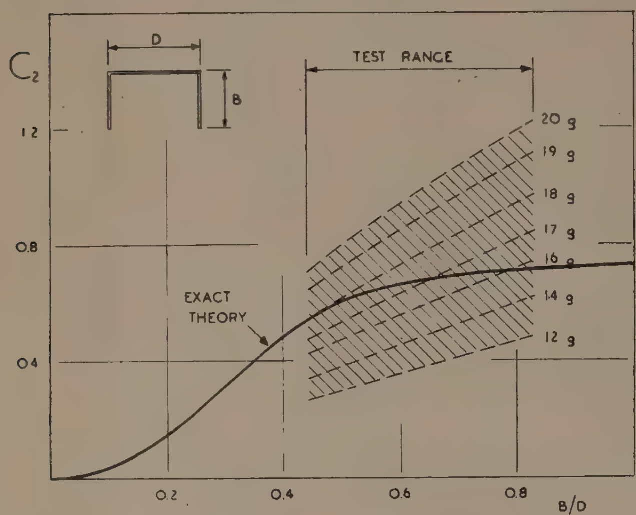
where, according to Lundquist, C_2 depends upon the geometry of the cross-section. For a plain channel section of uniform thickness, Lundquist found that C_2 is a function of B/D. For unstiffened plate components, the author suggests that

$$C_2 = \frac{B/t}{46 + .15B/t}$$

an expression which is quite independent of D. It would be interesting to know whether there is some theoretical basis for the author's expression, or whether it is purely empirical. If the author's expression is based on the earlier experimental work of Moir and Kenedi[17] (in which the dimension D for plain channel sections was maintained constant), then the author's expression, superimposed on the single curve of Lundquist, would appear as a family of curves corresponding to the various wall thicknesses. The experimental range of the tests of Moir and Kenedi for slenderness ratios not greater than 85 is roughly indicated in the illustration, Fig. 10. The compression is remarkable and may in fact suggest that the shaded portion representing approximately the field of validity of the author's expression corresponds to a scatter of experimental

results round the theoretical curve. The adoption of a single curve, such as that of the exact theory, would certainly lead to a considerable simplification of the whole problem of increased component plate strength.

Dr. R. M. Kenedi writes : Mr. Shearer Smith in his commentary on the proposed design specification refers to equality of overall column and component plate strength as the most desirable basis of efficiently econ-



$$f_c = \frac{C_2 E}{1-\sigma^2}\left(\frac{t}{B}\right)^2.$$

WHERE $C_2 = \dfrac{B/t}{46 + 15\,B/t}$

Fig. 10

omic design. The writer would like to support Mr. Shearer Smith most strongly on this point. Structural efficiency—primarily because of the use of a restricted and far too long unchanged range of "standard" sections—has always been a very neglected foster-child of present-day structural practice.

The enormous adaptability of the cold rolling process (within the limitations imposed by the overall width of the strip) permits the production of not only different sizes of the same section but of a range of sections of different proportions and shapes. In such circumstances no amount of attention paid to structural efficiency can be excessive.

Recently, factors influencing section efficiency have been the subject of some consideration[35] and it may be of interest to give here—possibly as an addendum to Mr. Shearer Smith's paper—a sample of an efficiency analysis, that of a plain column.

The problem briefly is as follows : What are the proportions of a plain channel section which will carry the greatest load per unit structural weight for a given strip width (i.e., circumference of section), thickness of strip and length ?

Assuming that torsional buckling does not occur, four modes of failure are relevant :—

(i) Overall column buckling about an axis through the centre of gravity and parallel to the flanges—axis XX.

(ii) Overall column buckling about an axis through the centre of gravity and parallel to the web—axis YY.

(iii) Plate component failure of the web—width W.

(iv) Plate component failure of the flange—width . b

Let the circumference of the channel be $c = 2b + W$

area $A = (2b + W)\,t = ct$

and $H = W/b$

Then the radii of gyration about axes XX and YY can be expressed as,

$$r_x = \frac{cH}{2(2+H)} \times \sqrt{\frac{H+6}{3H+6}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$r_y = \frac{c}{(2+H)^2} \times \sqrt{\frac{1+PH^1}{3}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Considering overall column failure and taking the critical stress f_c as given by equation (1) of the paper, the column loads relevant to failure modes (i) and (ii) become

$$P_x = f_c\,A = \frac{C_1\,E}{(1/r_x)^2} \times ct$$

$$P_y = f_c\,A = \frac{C_1\,E}{(1/r_y)^2} \times ct$$

Substituting for r_x and r_y from equations (1) and (2) and putting $P_x = P_y$ it can be shown that $H = .722$. This means that when the web is $.722 \times$ flange, the overall column strength of the plain channel is the same about both its axes. If H is less or greater than $.722$, P_x

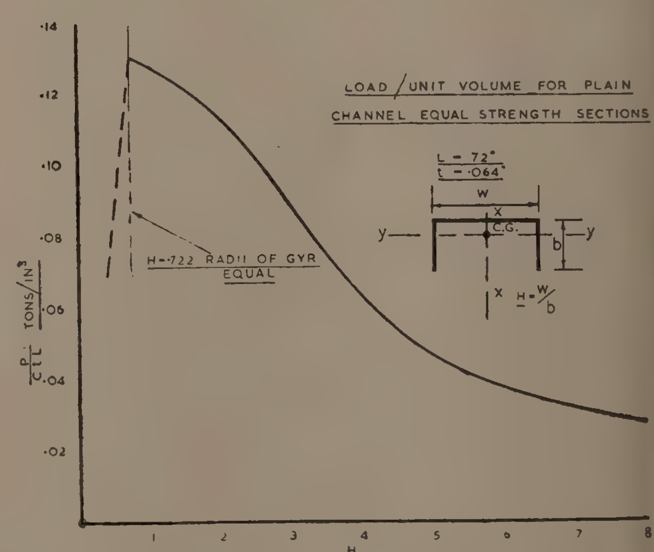


Fig. 11

is correspondingly less or greater than P_y. As it is always the lesser strength which defines the overall carrying capacity, it is obvious that as far as overall failure is concerned the maximum load that can be supported corresponds to $H = .722$.

By similar reasoning it can be shown that equal web and flange strength obtains when $H = 2.75$. This proportion gives the maximum load P which the column can support based on the plate component strength.

In an ideal case maximum efficiency obtains when all the possible modes of failure are equated. This obviously

cannot be done in the case of plain channels in view of the different values of H for overall and plate strength.

The next step is to decide whether it is preferable to work in the region of $H = .722$ or $H = 2.75$. This is achieved by investigating the load carried per unit structural weight (or load per unit volume) ratio for part "equal strength" sections lying between $H = .722$ and $H = 2.75$. The curves shown in Fig. 11 is the result of such an investigation for a constant length of 72 in. and a material thickness of .064 in. The values shown were obtained by equating the lesser column

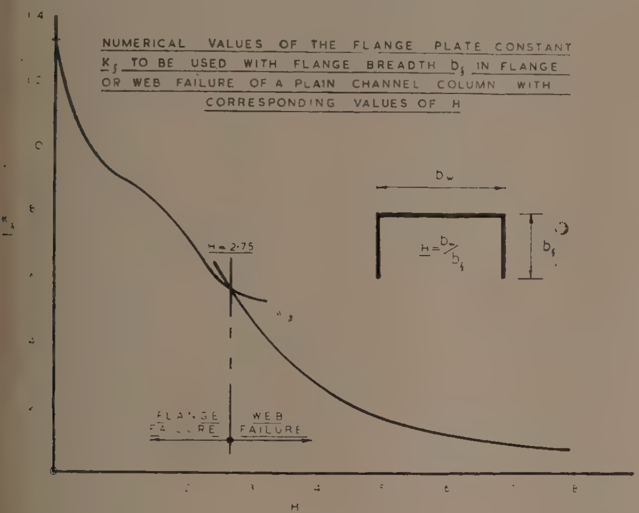


Fig. 12

strength (P_y) with the lesser plate strength (in this range the flange). The curve is extended to $H = 8$, the lesser plate strength from $H = 2.75$ to $H = 8$ being that of the web.

It is seen that at $H = .722$ the load per unit volume value is .13 against .095 at $H = 2.75$. That means that the structural efficiency of a plain channel column is about 1.37 times higher when the equality of failure modes (i), (ii) and (iv) is satisfied, than when column design is based on equality of failure modes (ii), (iii) and (iv). The load carried per unit volume drops away fairly rapidly after $H = 3$, reducing to about a quarter of that at $H = .722$.

From a practical point of view the results of the above enquiry mean that in plain channel columns, the web dimension should not be less than about $\frac{3}{4}$ and should not be more than about $2\frac{3}{4}$ of the flange width. This range of web to flange proportion ensures that the plain channel column is in the region of maximum structural efficiency.

Mr. JAMES M. HARVEY writes: Mr. W. Shearer Smith's proposed design specification is supported by experimental results and, while in his determination of permissible working stresses the minimum values of plate constants, etc., have been used, more exact analytical solutions for the interaction of plate components have been obtained, and some mention of these might be given here. For instance, for flexural instability of the flange of a plain channel section under compression, the value of the plate constant C_2 (as referred to by Mr. W. Shearer Smith in Clause 2 of the Commentary) was taken as 0.376, which is the value for a plate simply supported along one edge and free along the other edge, and is independent of the section shape. The correct value of C_2 lies between this and 1.092, the value for a plate fixed along one edge and free along the other edge, as the buckling plate, i.e., the flange plate, does receive some support from the web and the value of C_2 depends on the shape of the section.

In the analytical solution of this problem the method of integration of the differential equation for the deflected plate was used and the equation selected was the one employed by S. Timoshenko[36] in his buckling of uniformly compressed rectangular plates. If such an equation holds for the "buckling plate," i.e., the flange, a similar equation will be valid for the "supporting plate," i.e., the web, and if these two equations are taken together and solved, by equating the moments and slopes at the common corner, a solution is obtained in the form of a value of the plate constant C_2 for any ratio of the web to flange breadth. These values are shown in Fig. 12, where $K_f = C_2 \times \frac{12}{\pi^2}$ has been plotted instead of C_2. For $H = 0$ the value of K_f corresponds to a plate fixed and free along the edges and for $H = \infty$ the value of K_f corresponds to a plate simply supported and free along the edges. Mr. W. Shearer Smith's value of $C_2 = 0.376$, for a plate simply supported and free along the edges, which holds over the flange failure range is now of course $K_f = 0.376 \times \frac{12}{\pi^2} = 0.456$ and as can be seen from Fig. 12, this is very conservative, especially for lower values of H.

Depending on the section dimensions, it is possible to have the case of flexural instability of the web plate, and in this case the correct value of C_2 for the web lies between the minimum value of 3.291 for a plate simply supported along both edges and $C_2 = 5{,}758$ for a plate fixed along both edges. In the analytical solution we have an equation for the web or "buckling plate," and

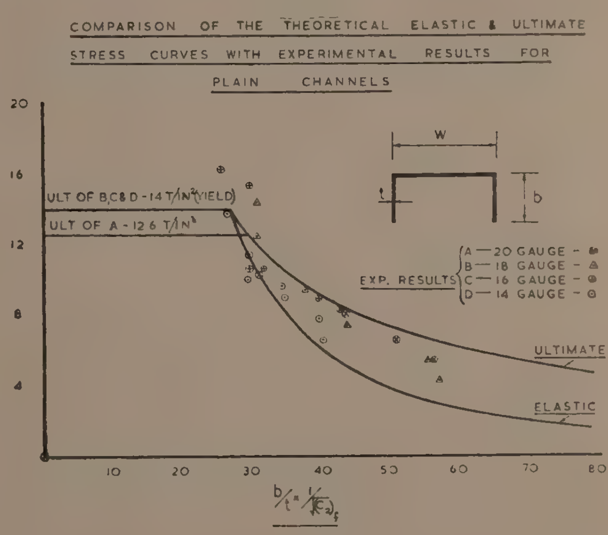


Fig. 13

one for the flange or "supporting plate," which this time gives a solution for the value of the plate constant C_2 for the web plate, for any ratio of the web to flange breadth. Since $K_f = Kw/H^2$, the Kw values can be converted to equivalent flange values, to be used in conjunction with flange widths and this has been done over the web failure range in Fig. 12. This allows the intersection of the theoretical curves for the different plate component failures to be obtained, indicating that at a web to flange breadth ratio of 2.75 equal web and flange strength obtains.

Using the same methods, an analytical solution for the buckling of the flange and web of inwardly and outwardly turned lipped channels can be obtained, assuming the lipped edge to be equivalent to a simply supported edge.

The analytical solution allows the elastic buckling stress for any section to be calculated and, as an example, for a set of plain channel sections, these are compared with the experimental results in Fig. 13. An ultimate stress curve based on the effective width conception, as given by S. Timoshenko,[37] utilising the same values of K_t already obtained for the elastic buckling stress is also included in Fig. 13 for comparison. The basis of plotting was selected to give single theoretical curves for all thicknesses in the instability range. The limiting stress for zero flange width is given by the yield stress value for the material or by the relevant ultimate stress for the web in the particular gauge, whichever is the smaller. It can be seen that reasonable agreement between experiment and theory is obtained and any scatter could be allowed for by fitting a Perry-Robertson type of transition curve to the lower scatter boundary of the experimental results.

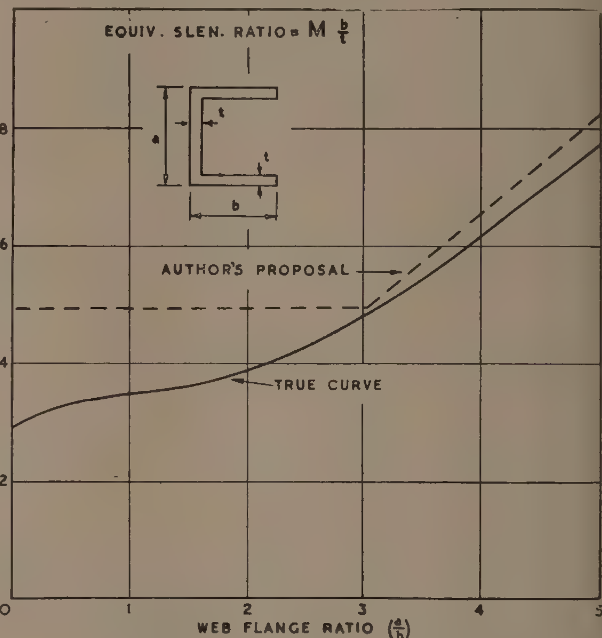
Mr. C. Marsh (Graduate) and Mr. J. B. Dwight write: Mr. Shearer Smith's admirable paper also has some bearing on light alloy design as there too the tendency is to use thin sections, and it is hoped that the following criticisms based on experience in this field will not be out of place.

For local buckling of struts (referred to as "plate component instability"), the author in effect treats an unlipped flange as a long plate free at one edge and simply supported at the other. For an equal angle this may be a reasonable approach. For a channel of squat proportions, however, it is unsound, as in this case the flanges cannot buckle without distorting the still stable web and thus receiving rotational restraint at the root. Table 1 is therefore conservative when applied to this type of section. The local buckling of unlipped channels and zeds has been thoroughly investigated by workers in the aircraft field independent enquiries in Great Britain[38], and America[39], leading to identical results, and there is no reason why this more accurate data should not be made use of. The local buckling strength of a channel may be found by first calculating the "equivalent slenderness ratio" R and then using this in conjunction with the Perry curve to obtain the critical stress R is given by :—

$$R = M \frac{b}{t}$$

where b and t are flange dimensions and M is a coefficient depending on the proportions of the section. A curve of M plotted against web-flange ratio is given in Fig. (14). This curve, which is based on the abovementioned aircraft data, enables a good estimate to be made of the local buckling strength of any unlipped channel (or zed). Although referred to flange dimensions, it is still applicable when web buckling governs failure (i.e., web-flange ratio over three to one), the value of M being adjusted accordingly when this is so. Denoting the web-flange ratio by N, the author's proposals (Table 1) amount to putting M equal to 4.95 or 1.65 N, whichever is the greater (see dotted curve in Fig. (14)). This leads to a low estimate of the critical stress, especially for low values of N. For example, the author would give the local buckling stress for a 3 in. $\times$ 2 in. $\times$ 14 SWG channel as 2.76 tons/sq. in.; this compares with the correct value of 4.17 tons/sq. in.

Lipped sections provide a more difficult problem, and it would be interesting to know what grounds the author has for considering that a $\frac{3}{8}$ in. lip is sufficient to provide a supported edge (Table 2). Surely the size of lip necessary to achieve this must vary with the size of flange. In the case of angles, the problem is most readily treated in terms of torsional instability. Channels are more difficult, and there is surprisingly little data available, although the work of Hu and McCullock[40] would suggest that a lip of 0.4 to 0.5 of the flange is needed in order to form a support. This is a lot greater than is convenient. There is no reason, however, why smaller lips should not be used, as the section will still be stronger than if unlipped, and it is suggested that for the time being a standard lip of one-fifth of the flange width should be adopted, the value of M for channels or zeds thus equipped being taken as three-quarters of the value obtained from Fig. 14. The results



LOCAL BUCKLING OF CHANNELS IN COMPRESSION

Fig. 14

of the above workers indicate that such a procedure would be safe. A more satisfactory rule can be formulated when further data becomes available.

In dealing with local buckling of the compression flange of a beam the author uses the same elastic formula (clause 3 (G) (iii) as he does for local buckling of struts. In bending, the tension flange has no tendency to buckle and restrains the rest of the section; the member is thus more stable than in direct compression where both flanges tend to buckle. The proposed code is therefore even more conservative for local buckling of beams than it is for struts.

Another shortcoming of the author's treatment is that whereas for local buckling of struts he uses a Perry type of formula, for all other forms of plate instability he uses the elastic formula unmodified, thereby failing to allow for imperfections and material failures which are known to bring the failing stress below the elastic figure.

The author states that for struts of channel section whose ends are prevented from relative rotation failure by torsion can be ignored. This is sound if the web-flange ratio exceeds a certain critical value (2.7 in the case of unlipped channels), but for sections more squat than this it is contrary to the accepted theory[41] and has been disproved by experiment[42]. With such sections torsional instability does matter, the range of slenderness ratio over which it governs increasing rapidly as the web-flange ratio decreases. The proposed code is seriously deficient on this point. For example, the top-hat rafter section in the roof truss—Fig. 7—is especially susceptible to mode of failure.

A final criticism concerns the extreme fibre stress in beams. The author suggests a permissible value of

$.65f_y$ in bending (clause 3 (a)) as against $.59f_y$ for axial loading. This increase may be reasonable for ordinary structural sections to BS.4, the shape factor of which is around 1.15, but for light-gauge members with a shape factor of virtually unity it is entirely unjustified.

Mr. K. C. ROCKEY writes : With reference to the design laws proposed in clauses 3 (c) and 4 (see Equations 9 and 10), it would appear to the writer that these are somewhat conservative. These design laws are obtained as indicated by the author, by applying a load factor of 2 with respect to the theoretical buckling values for simply supported edges. Since buckling under the action of shear or bending, or a combination of the two, does not usually lead to immediate collapse of the beam, but merely results in the formation of very shallow waves, with an accompanying redistribution of the stress system, it would seem that a load factor of 1.5 or 1.25 would have been satisfactory. The actual value chosen should depend upon the behaviour of the beam after buckling, and should be such that the ratio of the load required to cause the beam to cease to act as a normal load carrying member, to the proposed design loads, should lie within the range 1.5 to 2. If the beam is symmetrical, there would appear to be no reason, other than possibly that of flange flexibility, why the ratio-average stress at general yielding of the beam/buckling stress of web, should be unduly low. The writer would be glad if the author could indicate on what basis the load factor of 2 was chosen for these particular cases.

With reference to clause 5 (c) and Fig. 5, the writer would like to suggest that the following relationship, which is the equation of a quadrant of a circle with centre at *O.O.*, is more satisfactory and gives a closer approximation to the theoretical curve of Timoshenko than that proposed.

$$\left(\frac{f_s}{F^1{}_s}\right)^2 + \left(\frac{f_b}{F^1{}_{bc}}\right) = 1$$

hence

$$f_s = F'_s \left(1 - \left(\frac{f_b}{F'_{bc}}\right)^2\right)^{1/2}$$

Finally, in clause 4 (b), the author states that the proposed design stresses are for webs without stiffeners. Does he anticipate the use of intermediate stiffeners on these light gauge members, and if so, what correction to the proposed design laws would he consider suitable when such stiffeners are fitted, and also what specifications would he propose for the design of the intermediate stiffeners ?

Dr. C. M. MOIR (Member) writes : Two of the many commendable features inherent in this paper—all worthy of comment—are first, the close co-operation, extending over several years, that has been formed between an industry and the research team of a college. This is a partnership in which the practical engineer, while obtaining data from controlled experiments, allows the research expert to examine fundamental problems which must necessarily arise, and which may have a very wide and varied range of application. This fact differentiates between Industrial Research and an Industrial Investigation. The second notable aspect is that the research is continuing towards its ultimate goal which is the designer's drawing-board. Many valuable researches and data tend to fade out before reaching the engineer, who might have used the results to the benefit of industry.

This fine effort of presenting a paper in the form of a much-needed specification comes up against difficulties which are apparent. The specification is unbalanced in the sense that some sections have been written up after extensive tests ; other sections with their suggested stresses and ratios have been included after a limited number of tests, and the remaining sections include proposals apparently based on theoretical considerations. The whole scheme has been admirably sequenced after the style of B.S. 449. This does not appear to be quite logical. The B.S. 449 applies to hot rolled sections and has been compiled after many years of practical experience. The terms used in the specification are all well known. This proposed specification comprising new types of sections called " thin cold rolled " and demanding special techniques, and which are comparatively novel, must surely require a rather different type of specification. Terms such as " Lateral torsion bending strength under pure moment," and equations such as $f_c = \dfrac{1}{I_p}\left\{GJ + \dfrac{\pi^2}{l^2}(EC_{TB})\right\}$, not to mention square roots of fifth powers, are evidence of the academic mind and must be eliminated from a document which is essentially practical. This specification is in its infancy and must be carefully and fully presented. In the fulness of time and during years of experience there is no doubt it will be modified and probably changed considerably.

There is a tendency to over-emphasise the ideal of equal overall column and plate component strengths. In the testing of a single strut or a beam, this is a perfectly sound idea, but when that strut or beam becomes a part of a structure there are other considerations—the connections. It does not therefore follow that the ideal strut is the most economical. A structure composed of ideal members would not necessarily mean an attractive overall design and it would certainly be costly.

The author is, however, to be congratulated on the excellent and careful work he has expended in this very enterprising paper.

The AUTHOR replies : Before endeavouring to answer the various contributors, the author would like to take the opportunity of expressing his indebtedness to the Council for the honour conferred by them in awarding his paper the MacLachlan Lecture for 1950.

The author is appreciative of Professor Winters contribution, as it is well known that much of the progress of cold formed sections in America is due to the research work carried out under his direction.

Within the last few years correspondence has taken place with Mr. B. L. Wood, Consultant Engineer to the American Iron and Steel Institute, resulting in the author receiving from time to time valuable information including some of the references mentioned, but not all in time to be fully considered in the preparation of the paper, by then well under way.

It is agreed that there are several differences between the specification now proposed and the original American specification, where the use of design curves, unfamiliar in Great Britain, is widely adopted. The latter, in computing the properties of trial sections of flexural and compression members, uses a reduced " effective design width " in conjunction with a permissible stress based on a factored *collapse* stress, whereas the former believed to be more direct in its application, proposes the gross cross-sectional area multiplied by a variable permissible stress depending on l/r or b/t based on a factored *critical* stress.

Regarding the factor of safety, this is a matter which received careful consideration at the time and has since then been the subject of widely varying comment

from other contributors. In the material failure range the factor of safety proposed is slightly lower than that permitted in the American Specification, but in the plate component failure range the figure, based as it is on the "critical" stress, turns out higher than the American. It was considered unwise to condone the under-rolling of strip thicknesses by introducing a "factor of uncertainty," at so early a stage in the proceedings.

Professor Winter is correct in referring to the post-buckling strength of flat stiffened compression plates. It was intended that the specification should cover stiffened b/t ratios up to about 300 meantime, partly on account of lack of substantiating experimental data and partly due to the difficulties in rolling very wide flanges and webs perfectly flat. The use of very large b/t ratios in conjunction with the higher permissible stresses indicated by Professor Winter, results in overall and local distortions, a matter on which the consumer-public will have to be educated. In addition to the difficulty of convincing certain clients that members showing distortions are not on the point of collapse, there is the æsthetic aspect—one which cannot always be disregarded.

Much, if not all, Professor Winter says in his concluding remarks is true. Development in this country, although rapid in recent years, is still to some extent influenced by material and cold rolling plant available. The introduction of the new continuous sheet mill of Steel Company of Wales, however, should do much to improve the material position as regards both quality and quantity. On the other hand, it is relevant to point out that differences in the ratio of labour and material costs in the United States and United Kingdom respectively may easily lead to the structural development of cold formed sections in this country taking a different line from that in America. One may even venture to suggest that perhaps at a not too distant date American and British engineers will be referring to one specification acceptable to both.

Dr. Chilver, in raising three interesting points, contends in his first that partial equality of column and plate strengths (such as equality of column strength about one axis with one or both plate component strengths) does not necessarily lead to the most efficient form of column section—efficiency being defined as load carried per lb. weight of strut. In this he is correct. If it were possible to design a section with equality of strength in all modes of failure, i.e., equality in overall column failures and plate failures, maximum efficiency *would* result. This, however, cannot be achieved in the case of some cold formed sections (such as plain and lipped channels), on account of the uniform thickness of the material. It can be shown, however, that for sections not liable to torsion bending failure, the practically efficient design range, for example, for plain channels, lies between flange to web ratios of 1 to about 1/3, the efficiency for the square channels being somewhat higher. Thus a further example might be added to Dr. Chilver's table if a 2.25 in. × 2.25 in. × .104 in. (12g) channel is used for the 3-ton load over a length of 60 in. the working stress and area ratio would read 4.50 tons/sq. in. and .93 respectively. Incidentally, Dr. Kenedi's contribution throws considerable light on this point.

Regarding the increase in strength for plate components of a strut where there is no tendency to overall collapse Fig. 4, was based on empirical values where the limit of the experimental range was for a slenderness ratio not greater than 85. Dr. Chilver's opinion that this is of great practical importance confirms the author's own reason for the inclusion of this preliminary data which will in all probability vary with the degree o remoteness from overall column failure. Work, in whic Dr. Chilver has played a very prominent part, sponsore by the Cold Rolled Sections Association since the pap was submitted to the Institution, indicates that it possible to produce a transition curve which sati factorily takes this into account.

With reference to the third and last point, the ex pression for local instability stress is in fact an empiric expression. It was originally introduced to facilita direct design based on a single variable—the plate widt to thickness ratio. This has a definite advantage in th it allows the critical instability stress to be checked fo the plate components without reference to multi-curv charts necessary if C_2, made to vary with the flange t web ratio, is introduced. The author agrees fully wit Dr. Chilver that this is not exactly correct but conside it to be sufficiently accurate for purpose of illustratio and design.

It is doubtful if the discussion would have bee complete without Dr. Kenedi's contribution, as he ha been most prominent in the development of researc and design aspects of thin-walled sections in this countr Invaluable information is included for the designe giving the range within which he may expect to fin the most efficient section. In point of fact it woul appear that for each gauge there is one and only on most efficient strut of each type of section, plain channe lipped channel, etc. This does much to support th belief that wherever practical conditions permit, sectio can and should be designed and rolled to suit the pa ticular job and not just selected from a limited rang This is an important advantage which cold forme sections hold over hot rolled sections.

Mr. Harvey's contribution is also of particular interes as it gives analytical substantiation to the experiment data used in the preparation of the specification. Th fact that a theoretical solution has been evolved showin that the values of C_2 used in the paper are on the con servative side is most gratifying.

This would indicate that the permissible stresses i the plate failure range could be increased. It is poin such as this and, say, the question of factor of safet which will no doubt receive further attention prior t finalising the specification.

The contribution by Messrs. Marsh and Dwight appreciated, particularly as the author realises that th closer the co-operation between all users of light section whether alloy or steel, especially in research, will refle beneficially on the development of the appropria specifications.

In the case of plain channel sections which are stab as far as overall column buckling is concerned th mutual elastic restraint given by the web or the flang does give an increased strength to the critical com ponent. The form quoted in the paper, based o Lundquist's original work, applies to local buckling struts only. When, however, overall buckling is als present some allowance must be made for the lack o stability of the supporting plate. The figures given i Table 1 are on the conservative side—a point supporte by Messrs. Marsh and Dwight. The use of a transitio curve in this connection has already been mentioned i the reply to Dr. Chilver's contribution.

The minimum lip size of $\frac{3}{8}$ in. is an empirical figu and is restricted to the range of thicknesses quote Regarding the lip purely as an agency for providing edg support for the flange, it would appear that for practic purposes there is in fact a limiting lip size dependent o thickness. Further work on the subject would b beneficial.

As regards flange buckling of beams the conservative figures given for beams are partly due to the fact that at the time of preparation no theoretical information regarding the actual critical stress was available. Investigation of this is now proceeding at the Royal Technical College, Glasgow, and very recent theoretical work by Mr. Harvey will allow the revision of these figures in the near future. The reason for not using the Perry-Robertson type of formula for flange buckling was in order to make some allowance for the known conservative nature of the figures.

With reference to the point regarding torsional instability, the author fails to see where the value of 2.7 was obtained. Baker and Roderick in the reference quoted show a channel section of web to flange ratio of 2.2 to 2 which did not fail by torsional buckling but by flexural action. Analysing plain channels in the light of Baker and Roderick's findings by means of Timoshenko's general analysis (which incorporates Goodier's work) it can be shown that no torsional buckling failure occurs until the web to flange ratio is somewhat less than unity.

The reasoning with regard to the permissible extreme fibre stress in beams is not considered wholly correct, as the mechanism of collapse in thin-walled sections, although not entirely understood, is known to deviate from the basic concepts of the simple plastic theory. From experimental work carried out it was found that the values given by the simple plastic theory are always on the conservative side and that shape factors higher than 1.15 have been recorded in the material failure range of beam behaviour. The shape factor has no meaning in cases where failure is initiated by local instability, as in such cases the ratio of collapse moment to moment initiating elastic instability can have much greater values than unity, as for example five, as quoted by Professor Winter in his contribution. The figure suggested in the paper therefore is considered reasonable.

Turning now to Mr. Rockey's very welcome contribution, the author finds that the safety factors, considered too low by the previous contributors, are now thought to be too high. It is hoped that once the behaviour of thin-walled sections in bending is fully known and understood these load factors may be reduced to 1.5 and even 1.25, as suggested by Mr. Rockey. Meantime it is thought advisable to err on the safe side by using the value of 2 chosen from general structural experience.

Mr. Rockey's proposed alternative equation, for the Timoshenko combined bending and shear curve, is perfectly satisfactory, and could as he suggests be substituted for the expression quoted in the paper.

It is conceivable that intermediate stiffeners may be worthy of some research in the case of deep beams, but as the economics of their introduction as against, say, increasing the section thickness, is considered doubtful, the matter has not as yet been completely examined. Being aware of Mr. Rockey's first-hand knowledge on the subject of stiffeners, the author would be pleased to learn the results of his investigations.

Having known and had the privilege of working with Dr. Moir for many years, it is a particular pleasure to acknowledge his very pertinent and welcome remarks.

Speaking from the industrial side, the author fully endorses and bears witness to Dr. Moir's point about the benefits accruing from a full and free co-operation between industry and academic research teams.

The proposed specification is, as Dr. Moir suggests, to some extent unbalanced from the point of view of substantiating experimental data, but it should be borne in mind that research continues and that the "gaps" are being filled. It is not intended that the proposals put forward should be considered "the last word" but rather the first step towards an official specification. One reason for drawing up the specification on the familiar lines of B.S. 449 arose out of the well-known principle that new products sold in familiar wrappers are more likely, in the first instance, to find a consumer market than strange goods in even stranger wrappers. Cold formed sections properly applied are efficient media in certain fields of structural design and should, therefore, be treated as such. Many of the principles involved in design using hot rolled sections apply also to cold formed and to introduce unnecessary unfamiliar terminology would be a disservice to all concerned.

As for the terms quoted by Dr. Moir as being other than practical, may the author be permitted to point out that the first two appear only in the commentary, while the last is expressed similarly in B.S. 449. To remove all traces of the academic mind would result in only a collection of empirical rules remaining. Over-simple design rules, on the one hand, based on the superficial results of a limited range of experiments, can be very easily misapplied, whilst on the other a basic investigation using few assumptions usually results in rather complex formulæ which can only be presented for direct use by means of tables and charts. Surely the answer is to be sought somewhere between these two extremes.

Dr. Moir's suggestion that the importance of connections should not be lost sight of in the effort to design using only ideal sections is in large measure a wise one. The designer, however, who has a range of such sections to choose from is more likely to arrive at a more economical structure than, say, the designer who adopts only members which can be easily connected. One of the features of cold formed sections is that one section can sometimes be rolled to suit its neighbour, thus dispensing with gusset plates, etc. Careful consideration of the various aspects of each design is obviously advisable as well as desirable.

The author wishes to express his appreciation of the contributions received and trusts that he has covered all the points raised. The main objects of the paper were to stimulate interest in the use of comparatively new structural medium ; to make available to structural engineers and other potential users the basic data necessary to design and fabricate structures of cold formed sections ; and to bring to the notice of the appropriate authorities the need for a standard specification in this field.

In closing, may the author offer his sincere thanks, as a member of the Scottish Branch, to the President, Secretary and members of the Institution for a very enjoyable evening in London.

[35]Kenedi, R. M., and Harvey, J. M. "The Use of Equal Strength Sections in Structural Design." Transactions Institution of Engineers and Shipbuilders in Scotland, 1951.

[36]"Theory of Elastic Stability," by S. Timoshenko. Par, 65. P. 337.

[37]"Theory of Elastic Stability," by S. Timoshenko. Par. 73. P. 396.

[38]Royal Aeronautical Society (Data Sheets (Stressed skin)). 01.01.09.

[39]G. J. Heimerl—"Determination of Plate Compressive Strengths.—N.A.C.A. TN 1480. Dec. 1947.

[40]Hu and McCullock—"The Local Buckling Strength of Lipped zed Columns with Small Lip Width."—N.A.C.A. TN 1335. June, 1947.

[41]J. M. Goodiear—"Torsional and Flexural Buckling of Bars of Thin Walled Open Section under Compressive and Bending Loads." A.S.M.E. June 1942.

[42]Baker and Roderick—"The Strength of Light Alloy Struts." A.D.A. RR No. 3. July 1948.

Institution Notices and Proceedings

KING GEORGE VI

The Council of the Institution met on February 28th, and adjourned after recording their profound sorrow at the death of His late Majesty King George VI, and passing a resolution of sympathy with Queen Elizabeth, the Queen Mother, and the other members of the Royal Family.

ANNUAL DINNER

The Annual Dinner of the Institution, which was to have been held on March 20th, has been postponed and will now take place on October 2nd. Members will be notified of details in due course.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 28th, 1952, at 5.55 p.m., Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership ?

Students

BHATT, Amer Singh, of Brighton, Sussex.
BOTHMA, Kenneth Harold, of Johannesburg, South Africa.
BROWN, Leslie Arthur, of Trowell, Notts.
BURMAN, David Charles, of Liverpool.
BUTLER, Robert, of Stretford, Manchester.
CHAN WENG CHIU, of Singapore.
COBB, John Loraine, of Brighton, Sussex.
DAVIES, Norman Edward, of Rock Ferry, Cheshire.
IWANSKI, Zygmunt, of London.
LEADBEATER, Geoffrey, of Knutsford, Cheshire.
LEMPRIERE, John Victor, of Thornaby-on-Tees.
MEEKAN, John Byers, of Auckland, New Zealand.
MORLEY, Charles Edward, of Liverpool.
NEALE, David, of Swansea, Glam.
ROWLINSON, John Walter, of Romford, Essex.
SHERLOCK, William Albert, of Cardiff.
SHERMAN, Norman William John, of Surbiton, Surrey.
STUDZINSKI, Stanislaw Henryk, of London.
TAYLOR, Phillip Austin, of Brighouse, Yorks.
WILLES, Graham Cecil, of Little Chalfont, Bucks.

Graduates

BREARY, John Basil, of Petts Wood, Kent.
CANNING, Philip John Alexander, B.Sc.(Eng.), London, of Ilford, Essex.
CHATTERJEE, Binoy Kumar, of London.
CHURCHER, John, of Norwich.
DE PICCIOTTO, Maurice, B.Sc.(Civil) Birmingham, of New York, U.S.A.
DESHMUKH, Madhukar Dattatray, B.E.(Civil) Bombay, of Nasik City, India.
DIXON, Ronald John David, B.Sc.(Civil) Manchester, of Rhyl, Flintshire.
DONALD, Allan, B.Sc.(Hons.) Glasgow, of Wellington, C.1, New Zealand.
GAUNT, Grenville, of Chesterfield, Derbyshire.
HAWKER, Geoffrey Fort, B.Sc.(Eng.) London, of Woodford Green, Essex.
LAWRANCE, David Edward, A.M.I.Mun.E., of Shenley, Hertfordshire.
McKAY, Donald Crowther, B.Sc.(Civil) Cardiff, of Portreath, Cornwall.
McKENNA, Joseph Laurence, of Nantwich, Cheshire.
MASON, James Roy, of Surbiton, Surrey.
ONIONS, Richard Laurence, of Harrow, Middlesex.
POLAND, John Henry, B.E.(Civil) N.Z., of Auckland, New Zealand.
POTDAR, Maganlal Wanji, B.E.(Civil) Bombay, of Nasik, India.
SELFE, Michael Alwyn, of Lyndhurst, Hants.

Members

BATHO, Thomas, of Ewell, Surrey.
BURREN, William Henry, A.M.I.C.E., of London.
FISHER, Philip Arthur, of Cheam, Surrey.
HAWKINS, Laurence Edward, M.B.E., B.Sc.(Eng.) London, A.C.G.I., of Pinner, Middlesex.

Transfers

Students to Graduates

DARLISON, John Leonard, of London.
HUDSON, Hilbert Leslie, of London.
WALDEN, Kenneth Owen, B.Sc.(Eng.) London, of London.

Graduates to Associate-Members

DEMAIN, Richard Kenneth, of Prestwich, Lancs.
MALLICK, Samir Kumar, B.Sc., of Manchester,

Associate-Members to Members

ATKINSON, Alfred, A.M.I.C.E., of Rowley Bank, Stafford.
MARRIOTT, Edward Baxter, B.Sc.(Civil)S.A., A.M.I.C.E., of Pietermaritzburg, South Africa.
WILKIE, Basil, of Lagos, Nigeria.

Members to Retired Members

CROSS, Arthur George, L.R.I.B.A., of Natal, South Africa.
FORSTER, Thomas, M.B.E., of Ashford, Kent.
JACQUES, Herbert Hollings, B.Sc., A.R.C.Sc., of Dover, Kent.

Re-Admission

Associate-Member

BROOKS, Ivor, B.Sc., of Cape Town, South Africa.

OBITUARY

The Council regret to announce the deaths of Harbourne MACLENNAN, Allan RAMSAY MOON (Members); Henry Augustus MACKMIN (Retired Member) ; Henry Greville MONTGOMERY (Hon. Associate), and Edwin SMITH (Associate-Member).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of Stanley ALLEN-MAGILL, Stanley John DAVIES, John DOVASTON, James Alfred Henry HARPER, Francis Lee HOTHERSALL, William Hector MACKENZIE (Members) ; Arthur Octavius EDWARDS, James Hunter THOMSON (Retired Members) ; Terence CARR, Julian Granville CLAESSEN, Bertram Kimberley NICKLEN, Frederick William TREADWELL (Associates) ; Brian Mortimer ARCHIBALD, Horace BOULTON, John CROSBIE, Herbert Cutler WILKINSON, William Thomas WILKS (Associate-Members) ; Eric ADDICOTT, Philip Arthur BAYS, Thomas Egerton

BLAKEMAN, Hugh Noel Campbell MONCKTON, Eric SPENCER (Graduates) ; Miss Marjorie Amelia ALEXANDER, John LAMBERT. Ernest SHARPE, George Clifford WILLGRESS (Students).

EXAMINATIONS

The examinations of the Institution will next be held at centres in the United Kingdom and overseas on July 15th and 16th (Graduateship), and July 17th and 18th (Associate-Membership).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1 :—

Thursday, April 24th, 1952

Ordinary General Meeting at 5.55 p.m. This meeting, which is for the election of members and is open only to corporate members of the Institution, will be followed by an Ordinary Meeting at 6 p.m., when Mr. S. Mackey, M.E., Ph.D., A.M.I.C.E.I. (Associate-Member), and Mr. D. M. Brotton, B.Sc., Ph.D. (Graduate), will give a paper entitled " An Investigation of the Behaviour of a Riveted Plate Girder under Load."

Members wishing to bring guests to the Ordinary Meeting are requested to apply to the Secretary for tickets of admission.

Thursday, May 22nd, 1952

Ordinary General Meeting (for the election of members), 5.55 p.m.

Annual General Meeting, 6 p.m.

Thursday, June 26th, 1952

Ordinary General Meeting, 5 p.m.

INSTITUTION AWARDS

The Council have awarded the Bronze Sessional Medal for the best paper read before the Institution during the Session 1950-51 to Lt.-Colonel G. W. Kirkland and Mr. A. Goldstein, for a paper on " Design and Construction of a Large Span Prestressed Concrete Shell Roof."

OVERSEAS REPRESENTATIVE

The Council have appointed Mr. C. W. Hamann, M.I.C.E. (Member), to be the Institution's Representative in the South Island of New Zealand in place of Mr. D. Bruce Smith, who is leaving Christchurch.

REPRESENTATION

The Council have re-appointed Mr. Gower B. R. Pimm, M.I.C.E. (Past-President), as one of the Institution's Representatives on the Council for Codes of Practice for Buildings, for a further period of three years.

EXAMINATION FEES

The fees for the Institution's examinations, from and including those for January, 1953, will be upon the following increased scale :—

GRADUATESHIP

Students	£2 12 6
Non-members	£3 3 0

ASSOCIATE-MEMBERSHIP

Students and Graduates ...	£3 3 0
Non-members	£3 3 0

The exemption fee in both cases will now be £1 11 6.

YEAR BOOK AND LIST OF MEMBERS

The Year Book and List of Members for 1952 will go to press on July 1st, for publication in October, when a copy will be sent to all members of the Institution.

Members are requested to inform the Secretary of any alterations in titles, degrees or addresses, which have not already been notified, by June 15th, in order that such amendments may be included in the new edition.

SOCIETY OF CHEMICAL INDUSTRY

CONFERENCE ON APPLICATION OF RESEARCH

A whole day Conference on the application of the research reviewed at the Building Research Congress, 1951, is being organised by the Road and Building Materials Group of the Society of Chemical Industry, with particular reference to papers presented to Division 2 of the Congress.

Among the papers proposed for discussion are the following :—

" General trends in the applications of research in building materials."

" The extent to which concrete quality control is being applied and the results thereof."

" The use of lightweight concretes."

" The use of alternative varieties of timber, and the durability of flooring."

The Conference will be held on April 17th, 1952, at the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1, in two Sessions—10 a.m. to 12.30 p.m., and 2 p.m. to 5 p.m.

Those wishing to take part are invited to advise the General Secretary of the Society of Chemical Industry, 56, Victoria Street, London, S.W.1.

TRAINING GRANTS FOR ENGINEERS AND SURVEYORS

In consequence of an agreement recently reached by the National Joint Council for Local Authorities' Administrative, Professional, Technical and Clerical Services, a comprehensive scheme of financial assistance towards tuition and examination expenses of staff employed by local authorities is now in operation. The scheme includes staff undertaking courses of study for recognised qualifications as surveyors, engineers, architects and similar professional vocations.

In 1946, the National Joint Council negotiated a scheme whereby staff who successfully passed the intermediate examination of a recognised professional qualification (including that of the Institution of Structual Engineers) were awarded grants of £15 with a further £30 on passing the final examination. In 1949, the National Joint Council submitted a revised scheme whereby grants be awarded towards the costs of preparing for the recognised examinations (which include those of the Institution of Structural Engineers) and towards the actual expenses of taking the examination.

The new scheme provides for grants of 75 per cent. of the tuition fees, 75 per cent. of the registration and exemption fees, the full examination entry fees (first attempt), 75 per cent. of the expenditure incurred in travelling expenses (a) for course of study, and (b) for examinations, and 75 per cent. of the expenses incurred in securing practical training which is a condition precedent for entry to the examination.

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, at 6 p.m., on Tuesday, April 8th, when a lecture on " Bridges " will be given by Mr. H. Shirley Smith.

Hon. Secretary : D. B. Rogers, 4, Portland Rise, Finsbury Park, N.4.

INTERNATIONAL ASSOCIATION FOR BRIDGE AND STRUCTURAL ENGINEERING

As announced in the Journal last June, the International Association for Bridge and Structural Engineering will be holding its Fourth Congress in Cambridge and London from August 25th to September 5th, 1952.

The Association, whose headquarters are at Zurich, was founded in 1929 for the purpose of promoting international co-operation among scientists, engineers and manufacturers and the interchange of knowledge, ideas and the results of research work in the sphere of bridge and structural engineering in general, whether in steel, concrete or another material. The first Congress of the Association, in the organisation of which the Institution of Structural Engineers took a leading part, was held in Paris in 1932. Subsequent Congresses were held in Berlin (1936) and Liege (1948). This is the first time that the Congress will have been held in Great Britain and the organisers hope that it will receive the support of as many as possible who are eligible to participate in its proceedings.

The Chairman of the British Section of the Association is Mr. Ewart S. Andrews (Past-President of the Institution and Vice-President of the Association), who, with the assistance of an Organising Committee and other sub-committees is directing the organisation of the Congress, and a comprehensive programme of events has been arranged. During the period August 25th to August 30th, the Congress will be centred at Cambridge, where the programme will consist of technical meetings at which papers will be read and discussed, a Reception by the University, and visits to places of interest in and around Cambridge.

Each of the six themes forming the technical part of the Congress will be discussed at a special working session, and the conclusions drawn from the discussions will be included in the Final Report of the Congress. Papers have been prepared for each of the themes in the programme, and these will be included in the " Preliminary Publication " to be issued before the opening of the Congress. The authors of papers will be given an opportunity at the working sessions to introduce explanatory diagrams, but it will be assumed that the papers have been read previously by those wishing to join in the discussion. Each paper will be published in full in the language in which it has been prepared, followed by a summary in each of the official Congress Languages. The " Final Report " will contain contributions to the discussions at the working sessions of the Congress, together with the general conclusions reached.

The following are the themes for discussion :—Bases of Calculations and Safety Considerations ; Development of Methods of Calculation ; Metal Structures—Fundamental Principles ; Metal Structures—Practical Applications ; Fundamental Principles and the Properties of Concrete ; Current Problems of Concrete and Reinforced Concrete ; Prestressed Concrete.

On August 30th the Congress will move to London and from September 1st to 5th, various visits and tours to places in and around London will be held. There will be a Government Reception and an official banquet at Guildhall on September 1st. During this period three-day excursions are being arranged to places of engineering interest in Scotland, North Wales and South Wales.

The Lord President of the Council, the Rt. Hon. Lord Woolton, C.H., D.L., LL.D., has accepted office as the President of the Congress, which is being sponsored by the Institution of Civil Engineers and the Institution of Structural Engineers. The Presidents of the two Institutions, Mr. A. S. Quartermaine and Mr. Walter C. Andrews have been appointed as two of the Vice-Presidents of the Congress, and the Secretaries, Mr. E. Graham Clark and Major R. F. Maitland, as Joint Honorary Treasurers.

Any member of the Institution who wishes to attend the Congress must first be a member of the International Association and the British Section. The annual subscription is one guinea, and application forms, together with further details of the Association's activities, may be obtained upon application to the Secretary of the Institution, 11, Upper Belgrave Street, London, S.W.1.

On behalf of the Organising Committee this opportunity is taken to express sincere appreciation and thanks to the following firms and organisations who have generously made contributions towards the costs of the Congress, which to date amount to £12,000 :—

Acrow Engineers, Ltd., Association of Consulting Engineers, Automobile Association, Balfour Beatty and Co., Ltd., British Constructional Steelwork Association, British Iron and Steel Federation, British Welding Research Association, Simon Carves, Ltd., Cementation Co., Ltd., Cement & Concrete Association, Richard Costain, Ltd., Demolition & Construction Co., Ltd., Robert M. Douglas (Contractors), Ltd., J. B. Edwards & Co. (Whyteleafe), Ltd., Expanded Metal Co., Ltd., W. & C. French, Ltd., Franki Compressed Pile Co., Ltd., Higgs & Hill, Ltd., F. R. Hipperson & Son, Ltd., Holland and Hannen & Cubitts, Ltd., Holloway Bros. (London), Ltd., Holst & Co., Ltd., Institution of Structural Engineers, Keir & Cawder, Ltd., John Laing & Son, Ltd., Lever Bros. & Unilever, Ltd., Peter Lind & Co., Ltd., Sir Robert McAlpine & Sons and Associated Companies, Sir Alfred McAlpine & Co., Ltd., Mears Bros. (Contractors), Ltd., Mitchell Engineering, Ltd., F. Mitchell & Son, Ltd., F. G. Minter, Ltd., John Morgan (London) Ltd., W. Moss & Sons, Ltd., John Mowlem & Co., Ltd., C. A. Parsons & Co., Ltd., Rush & Tompkins, Ltd., The Rubberoid Co., Ltd., A. E. Symes, Ltd., Taylor Woodrow, Ltd., Trollope & Colls, Ltd., Twisteel Reinforcement, Ltd., Wates, Ltd., Wests Piling & Construction Co., Ltd., George Wimpey & Co., Ltd., Yorkshire Hennebique Contracting Co., Ltd.

Since the estimated cost of the Congress amounts to £15,000, additional contributions to the fund will be greatly appreciated, and should be sent to Major R. F. Maitland, at 11, Upper Belgrave Street, London, S.W.1

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Tuesday, April 29th, 1952

Professor J. A. L. Matheson, M.B.E., M.Sc., Ph.D. M.I.C.E. (Member), on " Plasticity and Structural Design," at the College of Technology, Manchester. 6.30 p.m.

Thursday, May 15th, 1952

Annual Business Meeting.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 28 Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

The Annual General Meeting of the Branch will be held at the James Watt Memorial Institute, Birmingham at 6.0 p.m., on Tuesday, April 29th, 1952, followed by a paper on " Piling in Engineering Construction," by Mr. J. Owen Lake, A.M.I.Struct.E.

Hon. Secretary : E. R. Deeley, A.M.I.Struct.E. Arranmoor, Adshead Road, Dudley, Worcs.

GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Wednesday, April 30th, 1952

Mr. S. M. Cooper (Associate-Member), on " Investigation of the Failure of the Tacoma Narrows Bridge " (Long. Version); " River to Cross " (wind tests on the Severn Bridge model). The meeting will be held at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 7.0 p.m.

Friday, May 30th, 1952

Short papers by members of the Section.

Hon. Secretary : M. H. Evans, B.Sc., 42, Church Hill Road, Handsworth, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The Annual General Meeting of the Branch will be held at the Neville Hall, Newcastle, at 6.30 p.m., on Wednesday, April 2nd, 1952, and will be followed by a paper on " Data in the Drawing Office," by Mr. J. Ross. The meeting will be preceded by tea at 6.0 p.m.

Hon. Secretary : Ian MacGregor, M.I.Struct.E., 9, Ellison Place, Newcastle-upon-Tyne, 1.

NORTHERN IRELAND BRANCH

The Annual General Meeting of the Branch will be held at the College of Technology, Belfast, at 7.30 p.m., on Tuesday, April 22nd, 1952.

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., " Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The Annual General Meeting of the Branch will be held at the Ca'doro Restaurant, Glasgow, at 6.0 p.m., on Wednesday, April 17th, 1952.

Hon. Secretary : D. G. Drummond, B.Sc., M.I. Struct.E., A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary : E. W. Howells, A.M.I.Struct.E., c/o Messrs. T. Harding & Sons, Ltd., 10/12, Market Street, Torquay.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Tuesday, April 1st, 1952

Students' Evening at the South Wales Institute of Engineers, Cardiff, at 6.30 p.m.

Friday, April 25th, 1952

Annual Dinner at the Osborne Hotel, Swansea.

Tuesday, May 6th, 1952

The Annual General Meeting of the Branch at the South Wales Institute of Engineers, Cardiff, at 6.30 p.m.

Saturday, June 7th, 1952

Joint visit with the Midland Counties Branch to the Penmaenmawr Welsh Granite Quarries and Llandudno.

Hon. Secretary : E. R. Steward, A.M.I.Struct.E., Edrom, Ashleigh Road, Blackpill, Swansea.

WESTERN COUNTIES BRANCH

The Fifth Meeting of the Session was held at Bristol University on Friday, February 1st, 1952, and took the form of a combined meeting with the South-Western Association of the Institution of Civil Engineers, Professor A. G. Pugsley, O.B.E., D.Sc., M.I.C.E., F.R.Ae.S. (Branch Chairman, Institution of Structural Engineers), presiding.

The paper entitled " Some Effects of Recent Developments on the Design and Construction of Concrete Structures," was presented by Dr. A. R. Collins, M.B.E., A.M.I.C.E., A.M.I.Struct.E.

An interesting discussion followed the paper, and a vote of thanks was proposed by Mr. Goodrich (Institution of Civil Engineers).

Friday, April 4th, 1952

The Annual General Meeting of the Branch, followed by a Film Show.

Hon. Secretary : C. E. Saunders, M.I.Struct.E., Dunkery, Edward Road, Walton St. Mary, Clevedon, Somerset.

YORKSHIRE BRANCH

The Annual General Meeting of the Branch will be held on Wednesday, April 23rd, 1952, at the Great Northern Hotel, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

Book Reviews

Law of Grading for Concrete Aggregates, by L. Boyd Mercer. (Melbourne : Melbourne Technical College Press, 1951). 113 pp. f'cap, numerous diagrams and photographs. Not priced.

Fuller's curve, modified by Boloney and others, remains the basis of acceptance of continuous gradings of concrete aggregates, and such gradings are often demanded without full appreciation of the cost. Mr. L. Boyd Mercer contends that, even if continuous gradings result in better concrete, there are many localities where they cannot be obtained economically and it is undesirable to insist upon their adoption. He has carried out an exhaustive programme of research on this important question of concrete economics, from which he concludes that mixes having discontinuous or gap gradings can present superior placeability and behaviour in comparison with those having continuous straight line gradings with the same water content, cement-aggregate and sand-stone ratios. In support of his argument the author presents the results of his experimental work in great detail, so that the reader can decide for himself whether or not he has substantiated his conclusions and his claim that his research confirms theoretical argument and M. Feret's observations that " as always, one rediscovers the laws of discontinuous mixtures which I proposed fifty years ago." If one criticism of a work which represents so much painstaking investigation may be permitted, it might be said that the argument is

obscured by the wealth of detail. A shorter version, containing just so much as is necessary to substantiate the author's conclusions, would be valuable as a warning against the blind acceptance of standard specifications.

R. V. C.

Fundamental Principles of Reinforced Concrete Design, by W. T. Marshall, Ph.D., M.I.Struct.E., A.M.I.C.E. (London and Glasgow: Blackie, 1951). 176 pp., $8\frac{1}{2}$ in. × 6 in. Price 20s.

In his foreword Professor Marshall makes it clear that he is producing a book mainly for students of Universities and technical colleges.

Having this fact in mind, the book is a most useful addition to those produced on R.C. design. It sets out in a clear and concise way the principles of design and develops the formulæ in stages which are easy to understand.

The practising engineer will find it quite refreshing reading, but he must not expect to discover the "aids to design" often used in the drawing office.

One small criticism is that in the "Steel Beam" method of reinforced concrete beam design no reference is made to the limitations of such a method.

A chapter is included on the plastic theory of design and another on prestressed concrete. These subjects are not treated in any great length, but an introduction is given to enable a student to understand the general principles. The bibliographies attached to these and other chapters are most helpful for following up purposes.

Fully worked out and practice examples, with answers, are well chosen and adequately illustrate the text. The book is an excellent production and can be recommended with confidence.

A. F. H.

Strength of Materials, by G. A. Olsen. (London: Allen & Unwin). $9\frac{1}{2}$ in. × 6 in. 442 pp. 32s. 6d.

Starting with elementary statics and simple stress systems, the author develops the theory of thin cylinders and spheres, the design of simple welded and riveted joints, and proceeds to shear-force, bending moment and deflection diagrams for simply supported, built-in and continuous beams. All deflection diagrams are calculated by area-moment method. There follows a section devoted to the design of columns. The book concludes with a treatment of combined stress systems and a note on stress concentrations and fatigue strength.

The subject is presented without using calculus. In avoiding the mathematical notation of calculus, the verbal exposition sometimes becomes rather complicated. In the treatment of slender columns the expression for Euler load is given without proof.

Some of the minor difficulties in understanding the text are no doubt due to the unfamiliarity of the American terminology, the expression for the solution of a quadratic equation, for example, is given (with a misprint) as the "Binomial theorem." It is perhaps more serious that examples worked to American Codes do not always conform to practice laid down in corresponding British ones.

The diagrams are excellent and many problems are given, some having answers supplied. As the book covers the fundamentals of the subject in a fairly descriptive way for the non-mathematically minded student, it is unfortunate that graphical methods find so little space in it.

E. R.

Advanced Strength of Materials, by D. A. R. Clark. (London: Blackie, 1951). 342 pp., $8\frac{3}{4}$ in. × 6 in. Price 35s.

This book, recently published, is written as a sequel to the author's "Materials and Structures," and is intended to cover Part II of the London University Engineerin Degree Examination in "Strength of Materials" well as various other University and Professional Exam inations of a similar standard.

It is divided into fourteen chapters dealing wi direct stress and strain; beams, encastre beams, co tinuous beams, stresses in beams; strain energy, torsio compound stress; thick cylinders; struts; curved bars stresses due to rotation, flat plates; vibrations an whirling of shafts.

Included in the text are more than 80 fully worke examples, whilst questions are set and answers given over 130 more. These constitute an invaluable aid the student whether working privately or in a colleg

Although written perhaps more from the standpoint the mechanical engineer than that of the structur specialist the latter will find much of interest, especiall in the chapters on beams and struts, strain energy an torsion.

The author and publishers are to be congratulated c the neat and clear diagrams and on the general attracti set-out of the vast amount of mathematical wo providing easy conditions for reading.

A. A. F.

Structural Theory, 4th Edition, by H. Sutherlan and H. L. Bowman. (New York: John Wiley, 1950 London: Chapman & Hall.) 394 pp., 9 in. × 6 in. 40

In the fourth edition of this book, a number of in portant additions have been made, particularly in th chapters dealing with slope and deflection, and rig frames, and much material has been rewritten.

The book deals primarily with the study of structur stress analysis and has chapters on graphic statics, ro trusses, truss and girder bridges, long-span bridge portal, rigid and space frames.

The historical development of the theory of structur is included, and the various methods of structur analysis now in use are described, including the Hard Cross column analogy method.

The book is valuable as a textbook for advance students and also as a reference book for practisin engineers.

An Introduction to the Design of Timber Struc tures, by Philip O. Reece. (London: Spon, 1949 235 pp., $8\frac{3}{4}$ in. × $5\frac{1}{2}$ in., Figures and Tables. 16s.

The wrapper states that Mr. Reece's book has bee written as a key to the information on Modern Timb Mechanics put at the disposal of the designer by researc institutions.

The author, in his preface, hopes that "the boo will interest engineers who know little about timbe timbermen who know little about engineering, an students who wish to learn something of both."

Personally, I expect to learn something of the desi of timber structures when I study a book so titled, an it seems to me that in keeping faith with the declaratio on the jacket and in his preface the author of this boo has devoted too much space to elementary theory structures of materials, data on various timbers, an (of interest in this connection only to a testing or researc engineer) statistical analysis. There is no one I kno more competent to write on timber design than M Reece, and I look forward to his next book, which trust will be so arranged and planned as to form compact reference book for the busy engineer, timbe man or student.

Design of Timber Structures contains excellent da on the design of connections, demonstrates most a the modifications of normal structural design necessa with timber as a medium, and a carefully referenced a exhaustive bibliography.

N.M.B.

The Analysis of Continuous Ridged Portal Frames

By E. Markland, B.Sc., A.M.I.C.E., A.M.I.Struct.E.

Summary

A method is presented for analysing portal frames of several continuous spans, attention being directed in the main to frames in which individual bays have symmetrical roof members.

The approach is essentially one of relaxation.[1] Rotational and linear movements are applied at the eaves joints so that, in the final condition, the out-of-balance thrusts and moments are negligibly small. The deflected shape of the frame under load is therefore obtained, and from this the bending moments round it.

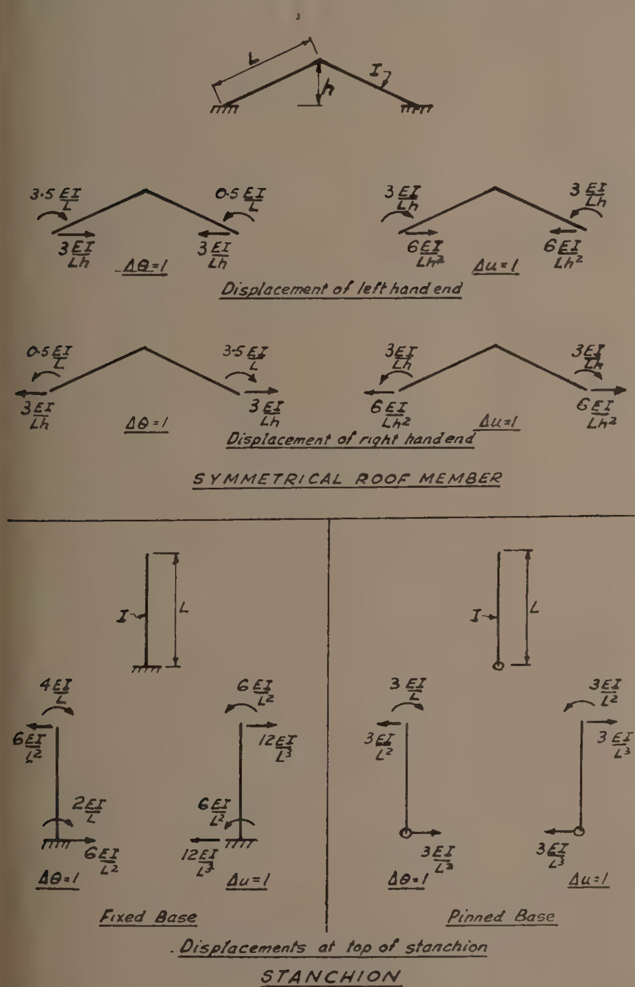


Fig. 1

A worked example of a 5-bay workshop-type portal-framed building under one condition of loading is given.

Introduction

As a preliminary to the analysis of a frame, consider the component members shown in *Fig.* 1. The roof member is symmetrical and the column may have either a fixed or a pinned foot ; each member had constant EI along its length.

We consider the moments and horizontal forces which are required to produce certain unit deformations of the members. Moments and angular displacements are reckoned positive when clockwise, and horizontal forces and linear displacements are reckoned positive when to the right. Unit angular deformation is denoted by $\triangle\theta = 1$ and unit linear displacement is denoted by $\triangle u = 1$. *Fig.* 1 shows the moments and forces which must be applied to the ends of members in order to produce unit displacements at either end of the roof member and at the top of the stanchion.

The derivation of each of these results is not given, as they may easily be obtained by, say, slope-deflexion analysis. For example, if F denotes the left-hand end of the roof member, O the ridge point and R the right-hand end, we have for $\triangle\theta = 1$ at the left-hand end,

$$M_{FO} = \frac{2EI}{L}(2+\theta_O)$$

$$M_{OF} = \frac{2EI}{L}(1+2\theta_O)$$

$$M_{OR} = \frac{2EI}{L}(2\theta_O)$$

$$M_{RO} = \frac{2EI}{L}(\theta_O)$$

$$M_{OF} + M_{OR} = 0$$

Elimination of θ_O gives $M_{FO} = 3.5\dfrac{EI}{L}$

and $M_{RO} = -0.5\dfrac{EI}{L}$

The horizontal thrust to the right at F is found to be $+\dfrac{3EI}{Lh}$.

In any problem the first step in the solution will be to substitute numerical values into the expressions given in *Fig.* 1 to obtain numerical values of the stiffness of members. Any multiplier to make the values of EI a convenient fraction of their actual values along corresponding members of the frame may be used.

The next step is to apply the load to the framework, considering the joints to be restrained against movement, so that fixed-end moments and forces are set up at the ends of loaded members. These moments and forces may be considered to be carried by the restraints at the joints, and since no movement of the joints has yet been considered to have taken place, no moments have yet been produced in members not carrying applied load.

Systematic displacements of the joints are now made in such a way that the moments and forces on the restraints are reduced to negligibly small values.

Moments round the frame are then calculated from the rotations and deflexions at the ends of the members.

Frame Analysis

Consider the frame shown in *Fig.* 2 to be divided into the basic components shown. Values of moments and horizontal forces required to produce unit displacements

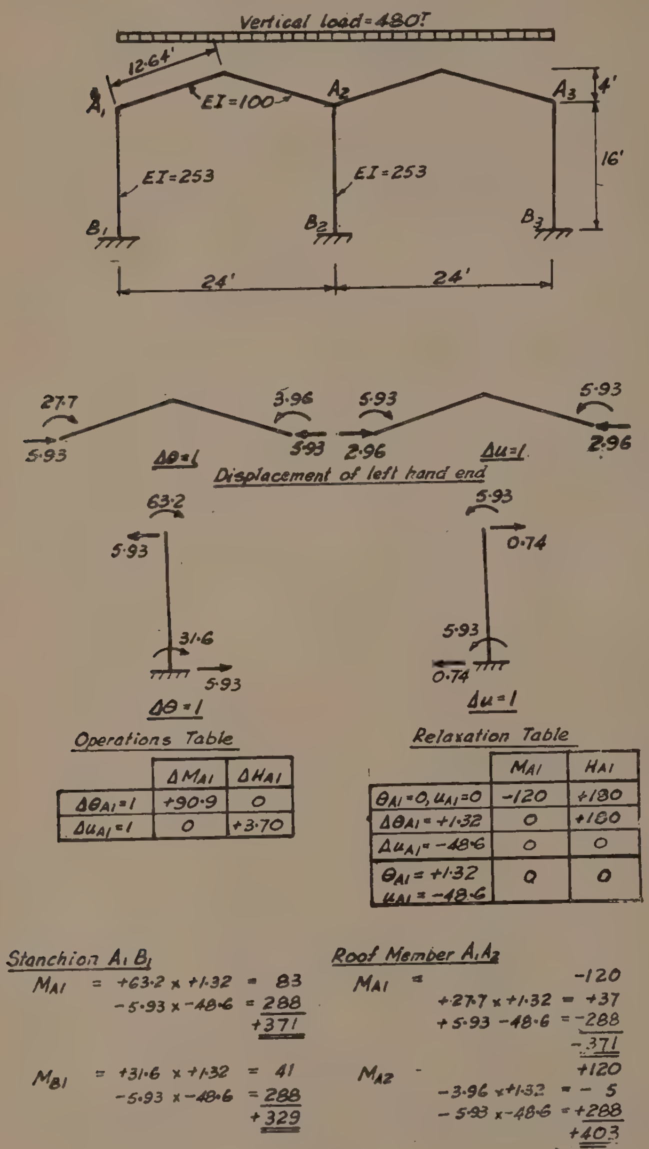


Fig. 2

at the ends of the roof members and at the top of the stanchions have been calculated by substitution into the expressions of *Fig.* 1.

A unit rotation at A_1 will require a moment of $+27.7$ at the end of A_1A_2 and a moment of $+63.2$ at the end of A_1B_1. The total moment required to rotate the joint A_1 will then be $+27.7+63.2=+90.9$. This unit rotation also requires a horizontal force of $+5.93$ on the end of A_1A_2 and a horizontal force of -5.93 on the end of A_1B_1; the total horizontal force is therefore $+5.93-5.93=0$. It is convenient to set out this information in an operations table, in the first line of which the operation representing a unit rotation $\triangle\theta_{A1}=1$ at A_1 is shown to require a change of moment at A_1 of $+90.9$ and zero horizontal force at A_1. The second line gives corresponding values for a unit displacement $\triangle u_{A1}=1$ of A_1 and is obtained by inspection of the forces and moments on the components shown in the figure.

If A_1 and A_2 were not allowed to move when the loading was applied, then $\theta_{A1}=u_{A1}=0$, and the moment M_{A1} at A_1 on the end of A_1A_2 would be the fixed end moment. The rafter length is 12 ft. (measured horizontally) and it is subjected to a vertical uniformly distributed load of 120 tons, so the fixed-end moment at A_1 is

$$-\frac{120\times 12}{12}=-120$$

The horizontal force to the right at A_1 may be found by taking moments for the rafter about the ridge, and is

$$\tfrac{1}{4}(120\times 12-120\times 6)=+180$$

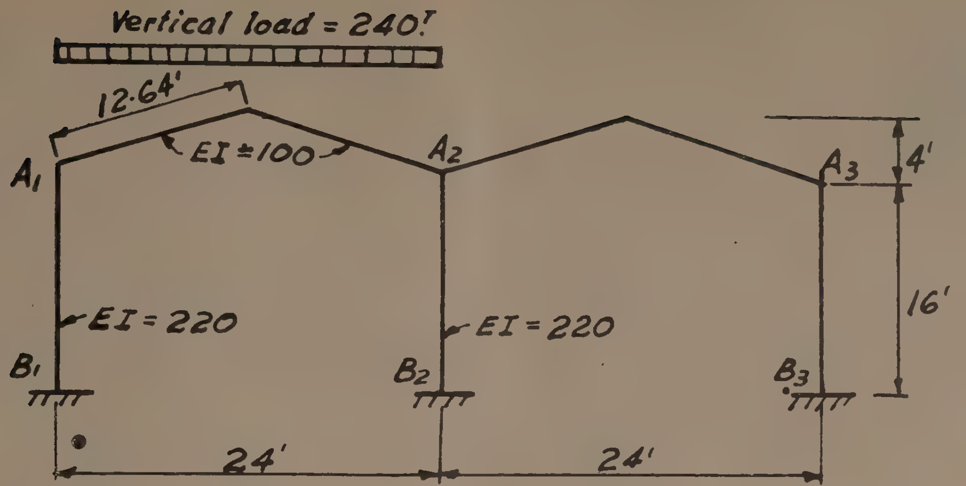
These values are set down in the appropriate columns in the first line of the relaxation table. It is our aim to reduce M_{A1} and H_{A1} to zero (or to negligibly small quantities) by imposing changes to θ_{A1} and to u_{A1}, i.e., by allowing A_1 to move from its initial position. The operations table indicates what changes should be imposed. $\triangle\theta_{A1}=+1.32$ produces $\triangle M_{A1}=+120$, and $\triangle u_{A1}=-48.6$ produces $\triangle H_{A1}=-180$; these are recorded in the lines of the table, current totals being kept in the M and H columns. The unbalanced moments and forces at A_1 have now been relaxed, that is reduced to zero, and moments at points in the frame are calculated from the known displacements and the known stiffnesses of the members.

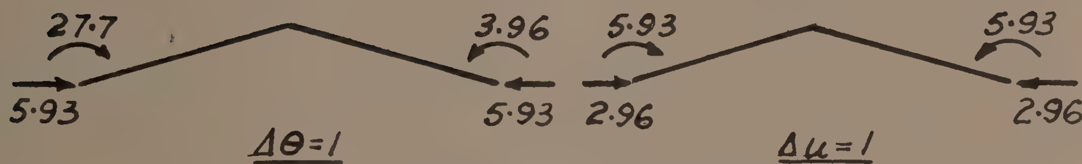
This example is particularly simple for two reasons. Firstly, the frame and loading are symmetrical; we can therefore say at the outset that $\theta_{A2}=u_{A2}=0$ and we do not consider displacements of A_2. Secondly, values of inertia of the members have been chosen such that the cross terms in the operations table are zero; having reduced M_{A1} to zero, the subsequent operation $\triangle u_{A1}=-48.6$ does not disturb it. In general this is not the case, and a converging series of successive approximations must be used to obtain a solution.

Neither of these simplifications exist in the frame shown in *Fig* 3. The operations table must therefore include six displacements (two at each of A_1A_2 and A_3). It is filled in by inspection of the figures immediately above the table. For example, if $\triangle\theta_{A2}=1$; $\triangle M_{A2}=+27.7\ +27.7\ +55.0=+110.4$, $\triangle M_{A1}=-3.96$, $\triangle M_{A3}=-3.96$, $\triangle H_{A2}=+5.93\ +5.93\ -5.16=+6.70$, $\triangle H_{A1}=-5.93$, $\triangle H_{A3}=-5.93$.

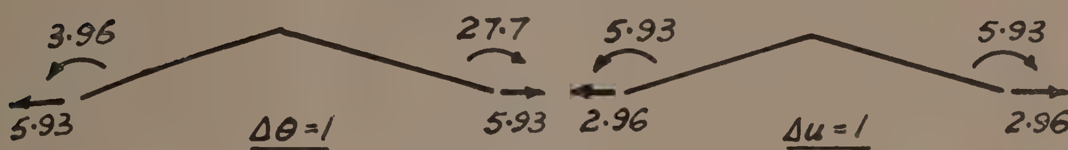
In addition to the six basic operations, the table shows the operation $\triangle u_{A1}=1$, $\triangle u_{A2}=-1$, acting simultaneously, and the block operation $\triangle u_{A1}=\triangle u_{A2}=\triangle u_{A3}=1$. The former group was formed because by inspection of the frame we see that the horizontal displacements at A_1 and A_2 will probably be about equal and opposite. The latter block operation will probably be useful to reduce the total horizontal force at eaves level to zero. Such group and block operation may be formed intuitively and shorten the relaxation table considerably; the block operation is likely to be valuable in any problem.

Fixed-end moments and forces have been entered in the first line of the relaxation table with the corresponding values of θ's = u's = 0. At (a) a group operation has been used; the magnitude being chosen to reduce H_{A1} to about zero. The rest of the table down to (b) represents relaxation by basic operations, current totals being kept in the table. At (b) it appears that all M's and H's are satisfactorily small, and the total current values of θ_{A1}, u_{A1}, etc., have been entered at (c) by addition of all the changes up to this point. *Using these values of θ's and u's the values of all M's and H's have been recalculated from the relevant coefficient*

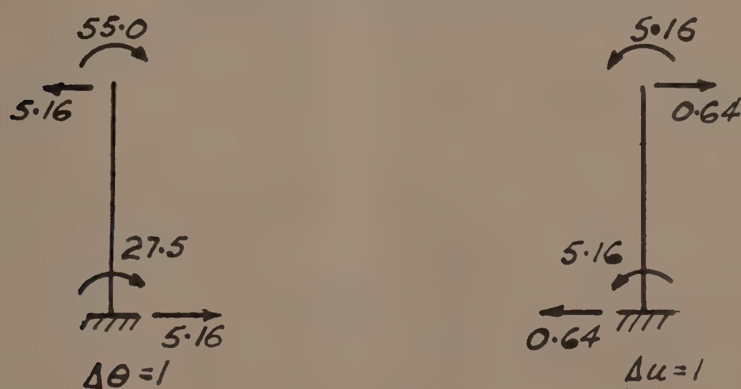




Displacement of left hand end



Displacement of right hand end



Operations Table

	ΔM_{A1}	ΔH_{A1}	ΔM_{A2}	ΔH_{A2}	ΔM_{A3}	ΔH_{A3}
$\Delta\theta_{A1} = 1$	+82·7	+0·77	−3·96	−5·93	0	0
$\Delta\theta_{A2} = 1$	−3·96	−5·93	+110·4	+6·70	−3·96	−5·93
$\Delta\theta_{A3} = 1$	0	0	−3·96	−5·93	+82·7	+0·77
$\Delta u_{A1} = 1$	+0·77	+3·60	−5·93	−2·96	0	0
$\Delta u_{A2} = 1$	−5·93	−2·96	+6·70	+6·56	−5·93	−2·96
$\Delta u_{A3} = 1$	0	0	−5·93	−2·96	+0·77	+3·60
$\Delta u_{A1} = 1$ $\Delta u_{A2} = -1$	+6·70	+6·56	−12·63	−9·52	+5·93	+2·96
$\Delta u_{A1} = \Delta u_{A2} = \Delta u_{A3} = 1$	−5·16	+0·64	−5·16	+0·64	−5·16	+0·64

Fig. 3 (1st Sheet)

Relaxation Table

	M_{A1}	H_{A1}	M_{A2}	H_{A2}	M_{A3}	H_{A3}	
u's $= \theta$'s $= 0$	−120	+180	+120	−180	0	0	
$\Delta u_{A1} = -30, \Delta u_{A2} = +30$	−321	−17	+499	+106	−178	−89	(a)
$\Delta\theta_{A2} = -4$	−305	+7	+57	+79	−162	−65	
$\Delta\theta_{A1} = +4$	+26	+10	+41	+55	−162	−65	
$\Delta\theta_{A3} = +2$	+26	+10	+33	+43	+3	−63	
$\Delta u_{A3} = +20$	+26	+10	−87	−16	+18	+9	
$\Delta u_{A2} = +3$	+8	+1	−67	+4	0	0	
$\Delta\theta_{A2} = +0{\cdot}5$	+6	−2	−12	+7	−2	−3	
$\Delta u_{A2} = -1$	+12	+1	−19	0	+4	0	
$\Delta\theta_{A2} = +0{\cdot}2$	+11	+1	+3	0	+3	0	
$\Delta\theta_{A1} = -0{\cdot}1$	+3	+1	+3	−1	+3	0	(b)
$\theta_{A1} = +3{\cdot}9$ $u_{A1} = -30$ $\theta_{A2} = -3{\cdot}3$ $u_{A2} = +32$ $\theta_{A3} = +2{\cdot}0$ $u_{A3} = +20$	+2·8	−0·2	+6·1	+2·3	+4·2	−1·7	(c)
$\Delta u_{A2} = -0{\cdot}4$	+5·2	+1·0	+3·4	−0·3	+6·6	−0·5	
$\Delta u_{A1} = -0{\cdot}3$	+5·0	−0·1	+5·2	+0·6	+6·6	−0·5	
$\Delta u_{A3} = +0{\cdot}2$	+5·0	−0·1	+4·0	+0·0	+6·8	+0·2	
$\Delta\theta_{A3} = -0{\cdot}08$	+5·0	−0·1	+4·3	+0·5	+0·2	+0·2	
$\Delta\theta_{A2} = -0{\cdot}04$	+5·1	+0·2	−0·1	+0·2	+0·4	+0·4	
$\Delta\theta_{A1} = -0{\cdot}06$	+0·1	+0·2	+0·1	+0·6	+0·4	+0·4	
$\Delta u_{A1} = \Delta u_{A2} = \Delta u_{A3} = -0{\cdot}9$	+4·7	−0·4	+4·7	+0·2	+5·0	−0·2	(d)
$\Delta\theta_{A3} = -0{\cdot}06$	+4·7	−0·4	+4·9	+0·6	0·0	−0·2	
$\Delta\theta_{A2} = -0{\cdot}05$	+4·9	−0·1	−0·6	+0·3	+0·2	+0·1	
$\Delta\theta_{A1} = -0{\cdot}06$	−0·1	−0·1	−0·4	+0·6	+0·2	+0·1	
$\Delta u_{A2} = -0{\cdot}1$	+0·5	+0·2	−1·1	−0·1	+0·8	+0·4	
$\Delta\theta_{A2} = +0{\cdot}01$	+0·5	+0·1	0·0	0·0	+0·8	+0·3	
$\Delta\theta_{A3} = -0{\cdot}01$	+0·5	+0·1	0·0	−0·1	0·0	+0·3	
$\theta_{A1} = +3{\cdot}78$ $u_{A1} = -31{\cdot}2$ $\theta_{A2} = -3{\cdot}38$ $u_{A2} = +30{\cdot}6$ $\theta_{A3} = +1{\cdot}85$ $u_{A3} = +19{\cdot}3$	+0·4	0·0	+0·1	+0·2	−0·1	+0·2	(e)

Stanchion A_1B_1

$M_{A1} = +55{\cdot}0 \times +3{\cdot}78 = +208$
$-5{\cdot}16 \times -31{\cdot}2 = +161$
$+369$ (369)

$M_{B1} = +27{\cdot}5 \times +3{\cdot}78 = +104$
$-5{\cdot}16 \times -31{\cdot}2 = +161$
$+265$ (262)

Stanchion A_2B_2

$M_{A2} = +55{\cdot}0 \times -3{\cdot}38 = -186$
$-5{\cdot}16 \times +30{\cdot}6 = -158$
-344 (347)

M_{B2} $+27{\cdot}5 \times -3{\cdot}38 = -93$
$-5{\cdot}16 \times +30{\cdot}6 = -158$
-251 (256)

Stanchion A_3B_3

$M_{A3} = +55{\cdot}0 \times +1{\cdot}85 = +102$
$-5{\cdot}16 \times +19{\cdot}3 = -100$
$+2$ (+7)

$M_{B3} = +27{\cdot}5 \times +1{\cdot}85 = +51$
$-5{\cdot}16 \times +19{\cdot}3 = -100$
-49 (48)

Roof Member A_1A_2

$M_{A1} = -120$
$+27{\cdot}7 \times +3{\cdot}78 = +105$
$+5{\cdot}93 \times -31{\cdot}2 = -185$
$-3{\cdot}96 \times -3{\cdot}38 = +13$
$-5{\cdot}93 \times +30{\cdot}6 = -182$
-369 (367).

$M_{A2} = +120$
$+27{\cdot}7 \times -3{\cdot}38 = -94$
$+5{\cdot}93 \times +30{\cdot}6 = +182$
$-3{\cdot}96 \times +3{\cdot}78 = -15$
$-5{\cdot}93 \times -31{\cdot}2 = +185$
$+378$ (382).

Roof Member A_2A_3

$M_{A2} = +27{\cdot}7 \times -3{\cdot}38 = -94$
$+5{\cdot}93 \times +30{\cdot}6 = +182$
$-3{\cdot}96 \times +1{\cdot}85 = -7$
$-5{\cdot}93 \times +19{\cdot}3 = -114$
-33 (29)

$M_{A3} = +27{\cdot}7 \times +1{\cdot}85 = +51$
$+5{\cdot}93 \times +19{\cdot}3 = +114$
$-3{\cdot}96 \times -3{\cdot}38 = +13$
$-5{\cdot}93 \times +30{\cdot}6 = -182$
-4 (−3)

Fig. 3 (2nd Sheet)

of the operations table and their initial values. For example,

$$
\begin{array}{llr}
M_{A1} & & = -120 \\
& +3.9 \times +82.7 & = +322.2 \\
& -3.3 \times -3.96 & = +13.1 \\
& -30 \times +0.77 & = -22.8 \\
& +32 \times -5.93 & = -189.7 \\
\hline
& & +2.8
\end{array}
$$

All calculations reported here have been done using a slide rule, but it is obviously quicker to perform this recalculation on a calculating machine. It will be noted that the new moments and forces differ from those in line (b). The discrepancies are due to rounding off errors when relaxing (no decimal figures have been used up to this point in the body of the table) and to arithmetical errors. It is the essence of relaxation method to execute the arithmetic as quickly as possible when relaxing and to make accurate checks such as line (c) as frequently as desired. The new values are now relaxed to line (e) where a final check shows very small remaining M's and H's.

The calculation of frame moments is shown at the end of the table. It is interesting to note that if values of θ's and u's from line (c) of the table had been accepted in view of the moderate size of M's and H's remaining at this point, the errors in frame moments would have been small. Values calculated from line (c) are shown in brackets on the figure. What, in effect, we are doing in accepting θ's and u's from line (c), is to calculate frame moments due to the specified loading less the loading represented by the values in the line, viz., $M_{A1} = +2.8$, $H_{A1} = -0.2$, etc., and it is obviously a matter of the designer's discretion to decide whether the error caused by this additional loading is of any practical consequence. For practical purposes the values in line (c) could be accepted with confidence.

In dealing with a frame such as that illustrated on *Fig.* 4, in which the stanchions have a change of inertia within their lengths, it is not possible to obtain values of stiffness of these members from *Fig.* 1. Reference may be made to charts[2] which give properties of such members, but in this example the necessary calculations by the method of column analogy have been set out. It is necessary to consider three conditions. Firstly, the moments and horizontal forces produced at each end of the stanchion by unit rotation at eaves level, secondly, moments and horizontal forces produced by unit sway without rotation at eaves level, and thirdly, fixed-end moments due to the applied loading. To avoid very small numbers in calculating the properties of the analogous column, an analogous column representing a stanchion with a roof leg of EI equal to 1, and with a crane leg of EI equal to 1/5 has been chosen for the calculation. The moments and forces required to produce unit deformations of this stanchion have been multiplied by 100 to obtain those required on the actual column, which has values of EI which are one hundred times larger.

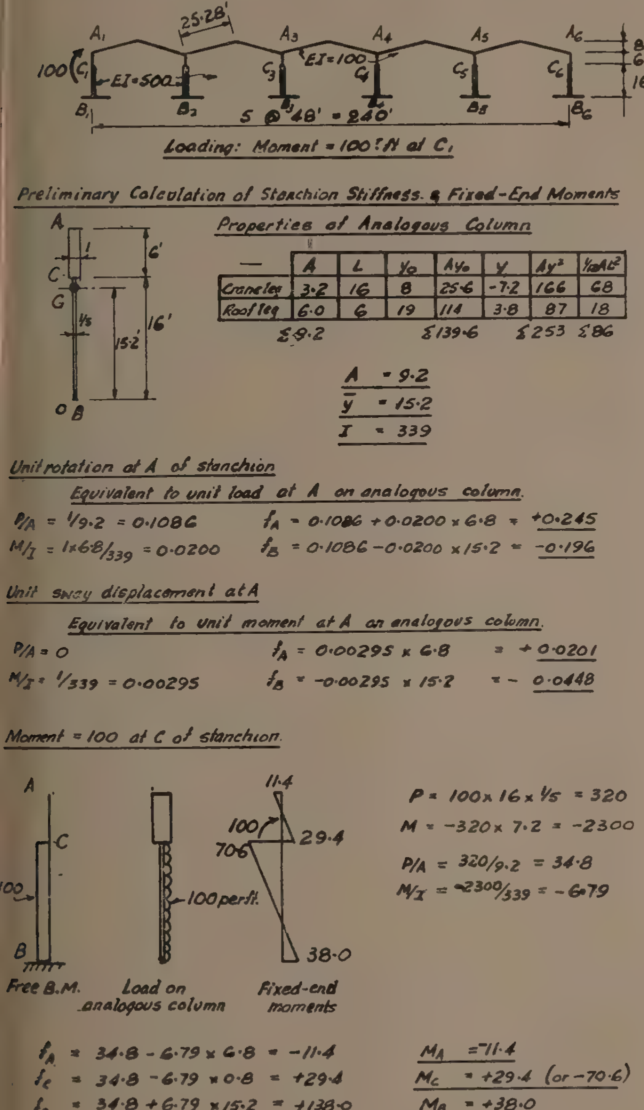
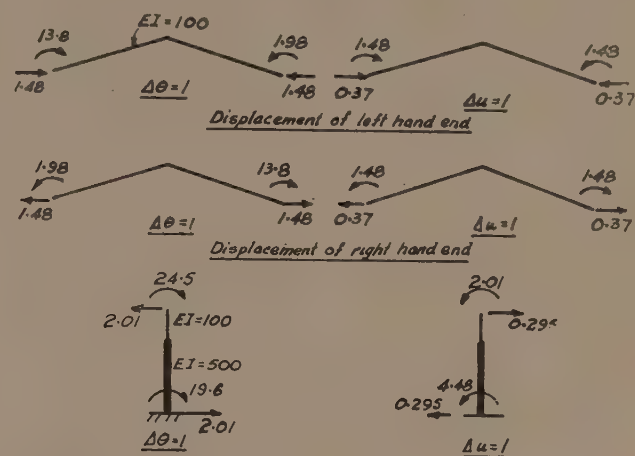


Fig. 4 (1st Sheet)



Operations Table

	ΔM_{A1}	ΔH_{A1}	ΔM_{A2}	ΔH_{A2}	ΔM_{A3}	ΔH_{A3}	ΔM_{A4}	ΔH_{A4}	ΔM_{A5}	ΔH_{A5}	ΔM_{A6}	ΔH_{A6}
$\Delta\theta_{A1}=1$	+38·3	−0·53	−1·98	−1·48								
$\Delta\theta_{A2}=1$	−1·98	−1·48	+52·1	+0·95	−1·98	−1·48						
$\Delta\theta_{A3}=1$			−1·98	−1·48	+52·1	+0·95	−1·98	−1·48				
$\Delta\theta_{A4}=1$					−1·98	−1·48	+52·1	+0·95	−1·98	−1·48		
$\Delta\theta_{A5}=1$							−1·98	−1·48	+52·1	+0·95	−1·98	−1·48
$\Delta\theta_{A6}=1$									−1·98	−1·48	+38·3	−0·53
$\Delta u_{A1}=1$	−0·53	+0·665	−1·48	−0·37								
$\Delta u_{A2}=1$	−1·48	−0·37	+0·95	+1·035	−1·48	−0·37						
$\Delta u_{A3}=1$			−1·48	−0·37	+0·95	+1·035	−1·48	−0·37				
$\Delta u_{A4}=1$					−1·48	−0·37	+0·95	+1·035	−1·48	−0·37		
$\Delta u_{A5}=1$							−1·48	−0·37	+0·95	+1·035	−1·48	−0·37
$\Delta u_{A6}=1$									−1·48	−0·37	−0·53	+0·665
$\Delta\theta=1$	+36·3	−2·01	+48·1	−2·01	+48·1	−2·01	+48·0	−2·01	+48·0	−2·01	+36·3	−2·01
$\Delta u=1$	−2·01	+0·295	−2·01	+0·295	−2·01	+0·295	−2·01	+0·295	−2·01	+0·295	−2·01	+0·295

Fig. 4 (2nd Sheet)

Relaxation Table

	M_{A1}	H_{A1}	M_{A2}	H_{A2}	M_{A3}	H_{A3}	M_{A4}	H_{A4}	M_{A5}	H_{A5}	M_{A6}	H_{A6}
$\theta's = u's = 0$	+11·4	−6·8										
$\Delta u_{A1} = +14$	+4·0	+2·5	−20·7	−5·2								
$\Delta\theta_{A2} = +0·4$	+3·2	+1·9	+0·1	−4·8	−0·8	−0·6						
$\Delta u_{A2} = +6$	−5·7	−0·3	+5·8	+1·4	−9·7	−2·8						
$\Delta u_{A3} = +3$	−5·7	−0·3	+1·4	+0·3	−6·9	+0·3	−4·4	−1·1				
$\Delta u_{A4} = +1$	−5·7	−0·3	+1·4	+0·3	−8·4	−0·1	−3·4	−0·1	−1·5	−0·4		
$\Delta u_{A5} = +0·4$	−5·7	−0·3	+1·4	+0·3	−8·4	−0·1	−4·0	−0·2	−1·1	−0·0	−0·6	−0·1
$\Delta\boldsymbol{\Theta} = +0·05$	−3·9	−0·4	+3·8	+0·2	−6·0	−0·2	−1·6	−0·3	+1·3	−0·1	+1·2	−0·2
$\Delta\theta_{A3} = +0·12$	−3·9	−0·4	+3·6	0·0	+0·2	−0·1	−1·8	−0·5	+1·3	−0·1	+1·2	−0·2
$\Delta\theta_{A2} = -0·07$	−3·8	−0·3	0·0	−0·1	+0·3	0·0	−1·8	−0·5	+1·3	−0·1	+1·2	−0·2
$\Delta\theta_{A1} = +0·1$	0·0	−0·4	−0·2	−0·2	+0·3	0·0	−1·8	−0·5	+1·3	−0·1	+1·2	−0·2
$\Delta\theta_{A4} = +0·03$	0·0	−0·4	−0·2	−0·2	+0·2	0·0	−1·2	−0·5	+1·2	−0·1	+1·2	−0·2
$\Delta U = +1$	−2·0	−0·1	−2·2	+0·1	−1·8	+0·3	−3·2	−0·2	−0·8	+0·2	−3·2	+0·1
$\Delta u_3 = -0·3$	−2·0	−0·1	−1·8	+0·2	−2·1	0·0	−2·8	−0·1	−0·8	+0·2	−3·2	+0·1
$\Delta\theta_{A4} = +0·05$	−2·0	−0·1	−1·8	+0·2	−2·2	−0·1	−0·2	−0·1	−0·9	+0·1	−3·2	+0·1
$\Delta\theta_{A6} = +0·08$	−2·0	−0·1	−1·8	+0·2	−2·2	−0·1	−0·2	−0·1	−1·1	0·0	−0·2	+0·1
$\Delta\theta_{A1} = +0·05$	−0·1	−0·1	−1·9	+0·1	−2·2	−0·1	−0·2	−0·1	−1·1	0·0	−0·2	+0·1
$\Delta\theta_{A3} = +0·04$	−0·1	−0·1	−2·0	0·0	−0·1	−0·1	−0·3	−0·2	−1·1	0·0	−0·2	+0·1
$\Delta\theta_{A2} = +0·04$	−0·2	−0·1	+0·1	0·0	−0·2	−0·1	−0·3	−0·2	−1·1	0·0	−0·2	+0·1
$\Delta\theta_{A5} = +0·02$	−0·2	−0·1	+0·1	0·0	−0·2	−0·1	−0·3	−0·2	−0·1	0·0	−0·2	+0·1
$\theta_{A1} = 0·20$ $u_{A1} = 15·0$ $\theta_{A2} = 0·42$ $u_{A2} = 7·0$ $\theta_{A3} = 0·21$ $u_{A3} = 3·7$ $\theta_{A4} = 0·13$ $u_{A4} = 2·0$ $\theta_{A5} = 0·07$ $u_{A5} = 1·4$ $\theta_{A6} = 0·13$ $u_{A6} = 1·0$	0·0	−0·1	+0·4	0·0	0·0	0·0	+0·6	−0·1	−0·1	0·0	+2·3	0·0
$\Delta\theta_{A6} = -0·06$												
$\Delta\theta_{A4} = -0·01$												
$\theta_{A1} = 0·20$ $u_{A1} = 15·0$ $\theta_{A2} = 0·42$ $u_{A2} = 7·0$ $\theta_{A3} = 0·21$ $u_{A3} = 3·7$ $\theta_{A4} = 0·12$ $u_{A4} = 2·0$ $\theta_{A5} = 0·07$ $u_{A5} = 1·4$ $\theta_{A6} = 0·07$ $u_{A6} = 1·0$	0·0	−0·1	+0·4	0·0	0·0	0·0	+0·1	−0·1	+0·1	+0·1	0·0	0·0

Fig. 4 (3rd Sheet)

The operations table now covers twelve displacements, and although it has been set out in full in the figure, it is obvious that for frames with many similar bays only the end and one typical internal joint need be written into the table. Block operations $\triangle\theta = 1$ representing unit angular displacement at all eaves joints and $\triangle U =$

representing unit linear displacement at all eaves joints together have been formed. The relaxation table is given in full as originally worked, although, when the M's and H's were recalculated, it appeared that mistakes have been made in relaxing M_{A4} and M_{A6}. The relaxation was continued to reduce these errors to negligible values, and the full bending moment diagram for the frame is shown in *Fig.* 5.

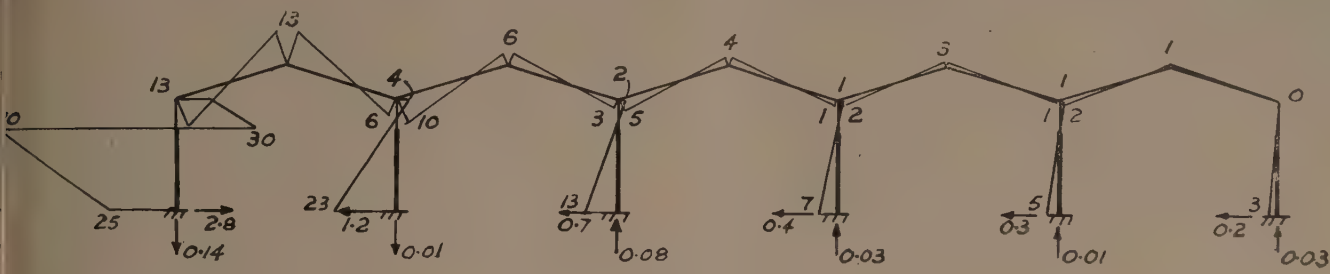


Fig 5

Examination of the earlier part of the relaxation table will show that where horizontal forces are being reduced from the left-hand end of the structure, they have not merely been reduced to zero, but over-relaxed from negative values to small positive values. For instance in the second line of the table $\triangle u_{A1} = +14$ has changed H_{A1} from -6.8 to $+2.5$, whereas $\triangle u_{A1} = +10$ would have changed it to -0.1. This apparently too large displacement is in anticipation of the subsequent change in H_{A1} when H_{A2} is relaxed in the fourth line of the table.

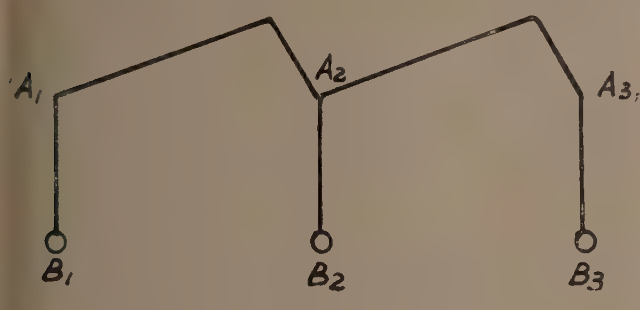


Fig. 6 (a)

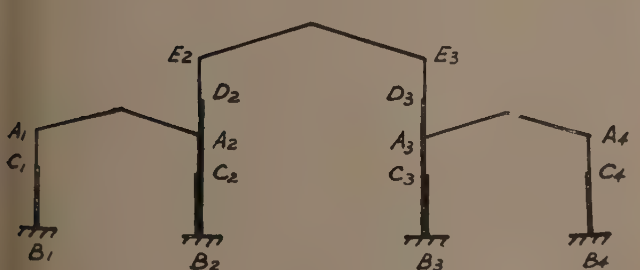


Fig. 6 (b)

The sum of corner moments and base moments is -97, which indicates an overall error of 3 in moments. This could be reduced by continuing the relaxation but in view of the loading condition this is hardly worth while. The load imposed by a crane wheel at C_1 will never be known in fact to an accuracy of better than 3 in 100, and, in any case, will include an arbitrary allowance of, say, one-fifth for impact. It may be noted that a force at eaves level of only 0.1 ton produces an overturning moment of over 2 ton-ft.

For frames with unsymmetrical roof members as shown in *Fig.* 6 (a) it is necessary to obtain the moments and horizontal forces which are required to produce unit deformations by a preliminary calculation, as the values given in *Fig.* 1 refer only to symmetrical roof members. (Similarly, if the roof is unsymmetrically loaded, fixed-end moments for the rafters must first be calculated.) Provided that the rib is made up of only two straight members it is probably most convenient to do this by the slope-deflexion method as the only two unknowns are the rotation and displacement at the ridge. For more complex ribs with several straight components or of the arched type, it is more convenient to use the method of column analogy to obtain the requisite preliminary information.

Fig. 6 (b) shows a typical frame which could be dealt with quite readily by the method described. Displacements of A_1, A_2, E_2, E_3, A_3 and A_4 would be considered. The members influenced by a displacement of A_2 would be A_1A_2, A_2B_2 and A_2E_2.

The Use of Models in Analysis

With the increasing use of complex continuous frames, analysis by models is becoming common to provide a check on, or to supplant, calculations. Models of complete multi-bay portal frames may be unwieldy ; a model of the 5-bay frame in which the stanchions were five inches high would be about 5 ft. long. The overall length of the model may, however, be reduced by distorting the horizontal scale.

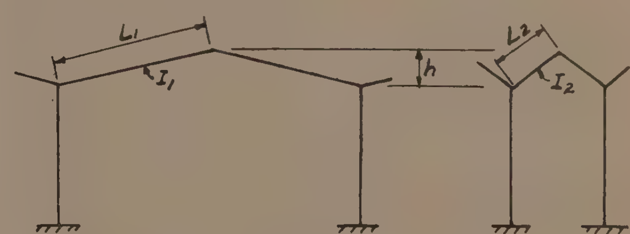


Fig. 7

Consider the two frames of *Fig.* 7 where the stanchions are identical but the symmetrical roof members, *although of equal rise h*, are of different spans and inertias.

$$\text{Suppose that } \frac{I_1}{L_1} = \frac{I_2}{L_2}$$

Then it will be seen by inspection of *Fig.* 1 that moments and thrusts required to produce unit deformations of the roof member will be the same in each frame. The behaviour of the frames below eaves level will therefore be identical and the model may be made of the more convenient second frame.

Suppose that a true scale model of the five bay frame had columns five inches high, rafter length of 5.75 inches, and bay spans of 10.9 inches, and the inertia of the rafter were represented in this model by a depth of 0.3 inches.

If, in the deformed model we wish for a span of 4.5 inches with the same column height, the deformed

rafter length would be 2.90 inches, and the required depth of rafter in this model would be

$$0.3 \times \sqrt[3]{\frac{2.90}{5.75}} = 0.24 \text{ inches}$$

The width of the vertical roof legs would, of course, remain at 0.3 inches. This model would be rather less than two feet long.

Conclusions

It will be seen that the relaxational approach described above provides an alternative method of analysing rigid frameworks of several bays, which, until the publication of Dr. Francis' paper[3] would have been very troublesome indeed.

The process converges rapidly since the effective carry-over factor of moments in the roof member is —1/7, and although the carry-over factor of horizontal forces is —1, these forces are rapidly absorbed by stanchions, which, in an actual building, are likely to be much stiffer than the roof members. The use of block displacement at eaves level also provides a convenient way of reducing total shear at eaves level to zero.

Relaxation method does not appear to have become quite so popular among practising structural engineers as its merit may suggest ; perhaps this is because there are already so many satisfactory methods available for analysis of single bay portal frames. With multi-bay frames the method does show to considerable advantage ; the completion of the 5-bay portal frame as far as the end of the relaxation table took less than two hours of calculation. The operations table, although perhaps appearing involved, can be written straight down once the properties of the components of the frame have been calculated. The relaxation table can be stopped when the fixed end moments and initial horizontal forces have been reduced to any desired amount ; this means that any desired accuracy may be obtained, so that in a tentative design made to check provisional scantlings a low degree of accuracy can be obtained quickly whereas in a final calculation the accuracy may be increased.

The fact that the calculations give the rotations and deflexions at eaves level is useful in cases where successive loadings are considered. In the example given above a solution for crane load on the end stanchion gave values of θ and u at the top of each of the stanchions. If we now consider the same frame with a similar unit loading at a penultimate column, the values already obtained may be used as a guide to provide an initial guess for rotations and deflexions in this case. Such a guess may be

$\theta_{A1} = 0.2$ $\theta_{A2} = 0.2$ $\theta_{A3} = 0.4$ $\theta_{A4} = 0.2$ $\theta_{A5} = 0.1$ $\theta_{A6} = 0.1$

$u_{A1} = 3$ $u_{A2} = 12$ $u_{A3} = 5$ $u_{A4} = 3$ $u_{A5} = 2$ $u_{A6} = 1$

and the relaxation started from this point.

The method may obviously be extended to deal with any system of connected arches, but in view of the popularity of ridged portals with symmetrical roofs some generality has been sacrificed to simplicity in the presentation.

Acknowledgements

The author is indebted to Mr. A. E. Holdsworth M.I.Struct.E., and to Mr. J. Springfield, B.Sc., for their criticism of the draft of this paper and their many helpful suggestions.

References

[1]Southwell, R. V. Relaxation Methods in Engineering Science. (Oxford University Press.)

[2]Markland, E. Moment-Distribution applied to Non-Prismatic Members. CONCRETE AND CONSTRUCTIONAL ENGINEERING, Vol. 43, No. 8, p. 226. (August, 1948.)

[3]Francis, A. J. The Analysis of Single-Storey Multi-Bay Gabled Rigid Frames. THE STRUCTURAL ENGINEER, Vol. 29, No. 7, p. 189. (July, 1951.)

A Theory for Struts with Lattice or Batten Bracing

By B. D Jones, A.M.I.Struct.E.

Summary

In this paper, it is assumed that the shear stiffness of a panel can be taken as distributed uniformly along the strut. "Secant" type formulæ (1), (2), (3) and (4) provide the crippling load of a strut with light shear bracing for the simpler case of equal eccentricities of end load ; formulæ (1), (4), (5) and (6) deal with the more general case of different end eccentricities and a uniform transverse loading. These formulæ differ from those for a strut with heavy shear bracing only in an extra term incorporating the shear stiffness. Expressions for shear stiffness of the usual forms of light shear bracing, such as lattice bracing, with and without posts, and batten bracing with equal and unequal chords, are given by formulæ (7) to (12).

Introduction

In some cases of strut design the load is comparatively small or the length comparatively long. Thus, if the design is based on a compact cross section such as a tube or "H" section, the slenderness ratio (L/K) of the strut will be high. The failing stress will tend to lie well down the Eulerian part of the strut curve and to be small compared with the yield stress.

A higher failing stress is obtained if the radius of gyration (K) can be increased. One method uses two light longitudinal members (chords), with much lighter transverse members whose function is :—

(a) to hold the chords apart so that a large value of "K" is achieved ;

(b) To form the shear connection between the chords, i.e., the bracing of the strut.

As is usual with members of a structure, there are both strength and stiffness requirements for the transverse members.

The bending moment in the strut, whether due to end moments or transverse loads, will vary along the strut. There must be shear force $\frac{dM}{dx}$ producing this variation. This shear force provides a strength criterion for the bracing. In the case of structural steelwork struts and columns used in buildings and bridges, there have long been empirical rules for the strength of lattice or batten bracing. For example, that the bracing should take a shear stress of 600 lb. per sq. in. or resist a shear force of .025 of the compression force. The case has also been discussed theoretically in ref. (1) ; though

order to find the shear force in steel columns with light bracing, the simplifying assumption was made that the effect of shear deformations can be neglected. Naturally these rules cannot be applied to other types of struts with light shear members. Other types include metal scaffolding ; the builder's or window-cleaner's ladder ; the plasterer's steps ; the boom of a jib crane ; water tower frames ; multi-storey single frame buildings without internal partitions, etc.

In addition, a stiffness criterion is necessary. Excessive shear flexibility increases the offset of the end load, which leads to an increase in the bending stresses in the chords (just as the lack of flexural stiffness does with a strut of compact cross section). The effect of shear deformation on the critical load of the *centrally* loaded column has been discussed as a case of elastic stability, Ref. 2, page 139. The result is strictly applicable to "long" columns only. For "short" columns, the Eulerian type of strut formula gives a critical stress greater than the yield stress of the material and this difficulty was overcome by replacing the modulus of elasticity by the reduced modulus, Ref. 2, page 196.

In the following, the critical load will be taken as that which produces the yield stress in some part of the column. This approach gives a "secant" type of formula, which is applicable to columns of any slenderness ratio.

NOTATION

A = area of cross section, of a prismatic bar or of *both* chords of a built-up column.

A_B, A_d, A_p, A_r, A_l = cross sectional area of batten, diagonal, post, right chord and left chord respectively.

a = spacing of the battens or posts of a built-up column.

b = distance between chords.

E = Young's modulus.

e_1, e_2 = eccentricities of applied end loads.

F = end load at any section along the column.

G = modulus of rigidity.

g = shear strain per unit force.

I = $\frac{1}{4} A b^2$ = moment of inertia of a built-up column.

I_B I_C I_l I_r = moments of inertia for batten, chord, left chord and right chord respectively.

k = proportion of shear taken by one (the left here) chord of a battened column with unequal chords.

M = moment at any section along the column.

P = applied end load.

p = is defined by equation (1), $p^2 = \dfrac{P}{EI(1 - gP)}$

Q = shear at any section along the column.

w = applied uniform lateral loading.

Z, Z_B, Z_C, Z_l, Z_r = section moduli of the column, batten, chord, left chord and right chord respectively.

Particular Case of Equal End Moments

Consider the column with pinned ends shown in Fig. 1. The eccentricity is the same at both ends.

To find the stresses in the column, the values of end load, shear force and bending moment are required at each section. The directions of M, F and Q shown in Fig. 2 are positive in sign relative to the axes in Fig. 1. The loads on the top part of the column are shown in Fig. 2. The equations of equilibrium are :

$$F \cos \theta + Q \sin \theta = P \quad \text{(a)}$$

$$Q = F \tan \theta \quad \text{(b)}$$

$$M = -P(\delta + e - y) \quad \text{(c)}$$

Making the usual assumption that the angle "θ" is sufficiently small to say that $\sin \theta$ and Q ($= F \tan \theta$) are small compared with $\cos \theta$ and F respectively, then $Q \sin \theta$ is negligible compared with $F \cos \theta$. Then,

$$F = P \quad \text{(d)}$$

$$Q = F \tan \theta = P \tan \theta = P \frac{dy}{dx} \quad \text{(e)}$$

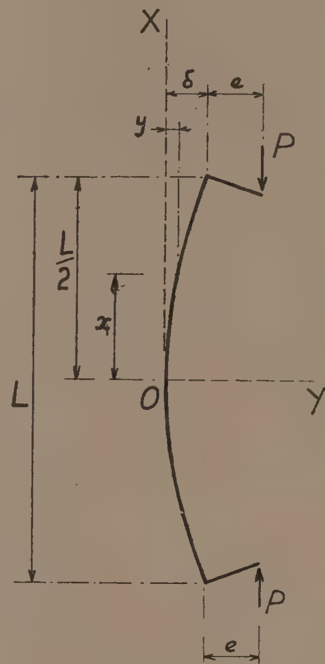


Fig. 1

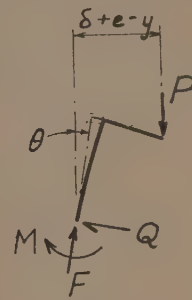


Fig. 2

Equations (c), (d) and (e) give the forces at the cut section in terms of the unknown quantity δ and the unknown variables y and $\dfrac{dy}{dx}$. To get any further we must consider the deflection curve. The curvature is due partly to the bending moment and partly to the shear force.

The additional slope of the deflection curve due to the shear force is seen, from the detail at the cut section Fig. 3, to be the shear strain, which will be denoted by gQ, where the shear force Q varies along the column. The term "g" is the shear strain due to unit shear force. It is constant for prismatic bars and when the column members are uniform in size and spacing. Its value depends on the type of structure used to carry the shear and on the stiffnesses of the shear members. Formulas for evaluating "g" are given later.

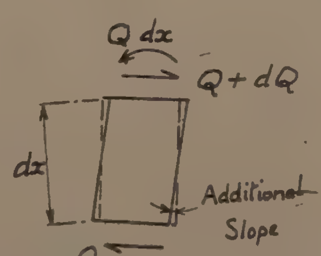


Fig. 3

The additional curvature due to the shear force is the rate of change of the additional slope. Adding

this to M/EI, the curvature due to bending only, gives the total curvature :

$$\frac{d^2y}{dx^2} = -\frac{M}{EI} + \frac{d}{dx} gQ \quad . \; . \; . \; . \; . \; . \quad (f)$$

The sign conventions in this section are the same as those on pages 71, 135 and 239 of ref. (7).

Substituting from (c) and (e) and rearranging,

$$\frac{d^2y}{dx^2} = \frac{P}{(1 - gP)EI} (\delta + e - y) \quad . \; . \; . \; . \quad (g)$$

The solution of this differential equation is the equation of the deflection curve,

$$y = (\delta + e)\,(1 - \cos px) \quad . \; . \; . \; . \; . \; . \; . \quad (h)$$

$$\text{where } p^2 = \frac{P}{EI(1 - gP)} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (1)$$

It can be shown that this is a solution by differentiating twice and then substituting for d^2y/dx^2 and y in equation (g). This is the particular solution because it satisfies the end conditions that both $y = 0$ and $\frac{dy}{dx} = 0$, at $x = 0$.

Putting $x = \frac{L}{2}$, the half length, gives

$$\delta = (\delta + e)\,\left(1 - \cos\frac{pL}{2}\right)$$

$$\delta - e \frac{1 - \cos pL/2}{\cos\frac{pL}{2}} \quad . \; . \; . \; . \; . \; . \; . \; . \quad (i)$$

The forces at the cut section are now determined. The bending moment, from equations (c), (h) and (i), is

$$M = \frac{-Pe}{\cos\frac{pL}{2}} \cos px \quad . \; . \; . \; . \; . \; . \quad (2)$$

The shear force at the cut section is, by differentiation,

$$Q \quad p \frac{Pe}{\cos\frac{pL}{2}} \sin px \quad . \; . \; . \; . \; . \; . \; . \quad (3)$$

The end load at the cut section is given by (d), i.e., $F = P$.

The extreme fibre stress due to end load and bending moment, at any section along the column is

$$f = \frac{P}{A} + \frac{M}{Z}$$

$$f = \frac{P}{A} + \frac{Pe}{Z\cos\frac{pL}{2}} \cos px \quad . \; . \; . \; . \; . \; . \quad (4)$$

where $Z = \frac{1}{2} Ab$ for the type of strut considered here. The maximum value of stress is when $\cos px = 1$, i.e., at $x = 0$.

The stress due to the shear force cannot be given in a form applicable to all types of column bracing. Thus, for a prismatic bar, the average shear stress is Q/A ; but with lattice braced columns for example, the shear force gives direct stresses in the bracing members. This is treated in detail later.

The Pinned Strut with Unequal End Eccentricities and Uniform Lateral Loading

The quantities e_1, e_2, w, y and z are positive as shown in Fig. 4.

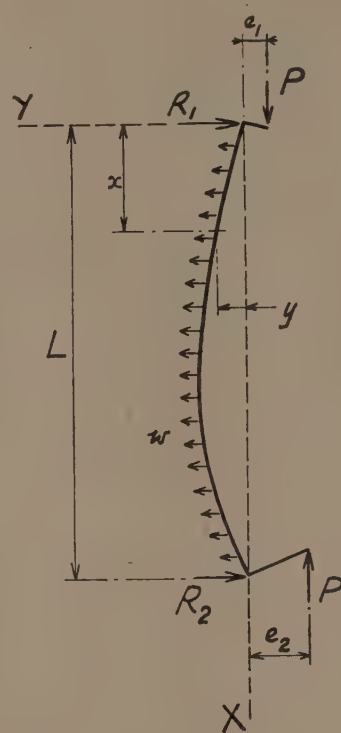


Fig. 4

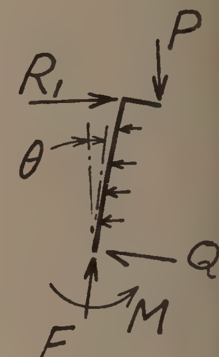


Fig. 5

From the equations of equilibrium of the strut, Fig. (4),

$$R_1 = \frac{wL}{2} + \frac{P\,(e_2 - e_1)}{L}$$

From the equilibrium of the top part, Fig. (5),

$$Q \cos\theta - R_1 + wz = F \sin\theta$$

$$M = P\,(e_1 + y) + R_1 - \frac{wz^2}{2}$$

Substituting for R, etc.,

$$Q - \frac{wL}{2} - \frac{P\,(e_2 - e_1)}{L} + wz = P\frac{dy}{dz}$$

Differentiating.

$$\frac{dQ}{dz} = -w + P\frac{d^2y}{dz^2}$$

As before

$$\frac{d^2y}{dz^2} = -\frac{M}{EI} + \frac{d}{dz}(gQ)$$

Substituting for M and $\frac{dQ}{dz}$, and rearranging,

$$\frac{d^2y}{dz^2} + p^2y = -p^2\left[\left(e_1 + \frac{gwEI}{P}\right) + \left(\frac{wL}{2P} + \frac{e^2 - e_1}{L}\right) z - \frac{wz^2}{2P}\right]$$

The solution of this differential equation satisfying the end conditions that $y = O$ at $x = O$ and $x = L$ is the deflection curve.

$$y = e_1 \left[\frac{\sin p\,(L - z)}{\sin pL} - 1 + \frac{z}{L} \right] + e_2 \left[\frac{\sin pz}{\sin pL} - \frac{z}{L} \right] + \left(\frac{w}{p^2 P} + \frac{gwEI}{P} \right) \left[\frac{\sin p\,(L - z)}{\sin pL} + \frac{\sin pz}{\sin pL} - 1 \right] - \frac{wz}{2P}(L - z)$$

The expression for the deflection curve when shear deflection is neglected is given in ref. (6), page 347. The difference here is only in the additional term $\frac{gwEI}{P}$ and in the modified value $p \doteq \frac{P}{(1 - gP)EI}$. The usual expression can be obtained by putting $g = O$, i.e., when the shear strain is negligible.

After substituting and simplifying, we can write the bending moment at any point as,

$$M = P \left[e_1 \frac{\sin p\,(L - z)}{\sin pL} + e_2 \frac{\sin pz}{\sin pL} \right] + \left(\frac{w}{p^2} + gwEI \right) \left[\frac{\sin p(L - z)}{\sin pL} + \frac{\sin pz}{\sin pL} - 1 \right] \quad . \; . \; . \; . \; (5)$$

The shear at any point is,

$$Q = Pp \left[\frac{-e_1 \cos p\,(L - z) + e_2 \cos pz}{\sin pL} \right] + p \left(\frac{w}{p^2} + gwEI \right) \left[\frac{\cos p(L - z) - \cos pL}{\sin pL} \right] \quad (6)$$

Prof. A. G. Pugsley has suggested deducing these results direct from the generalised theorem of three moments, i.e., generalised to allow for end loads and shear effects.

Formulae for "g" the Strain due to Unit Shear Force

Prismatic Bars

In the case of columns made of prismatic bars, the effect of shear deflection on the column strength is negligible. This is due to the comparatively thick sheet web. Typical formulæ are given only for the sake of completeness ; the method is in ref. (3).

For a circular cross section, $g = \frac{1.11}{AG}$ . . . (7a)

For a rectangular cross section, $g = \frac{1.20}{AG}$ . . . (7b)

For an "I" section with the shear acting in the plane of the web,

$$g = \frac{1.0}{(\text{Area of web only})\ G} \text{ approximately} \quad . \; (7c)$$

Lattice Braced Columns

It has been shown, ref. 2, page 143, that with "N" bracing Figs. 6, 7 (a),

$$g = \frac{1}{A_d E \sin \phi \cos^2 \phi} + \frac{1}{aA_p\ E} \quad . \; . \; (8)$$

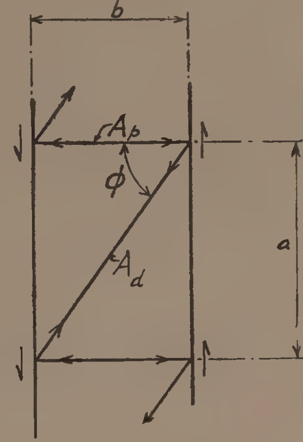


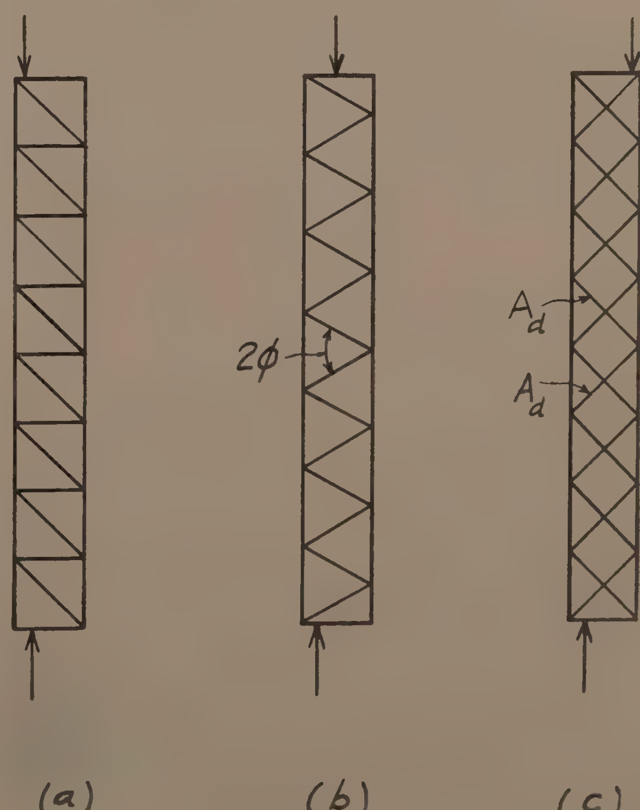
Fig. 6

When there are no posts, Fig. 7 (b),

$$g = \frac{1}{A_d E \sin \phi \cos^2 \phi} \quad . \; . \; . \; . \; . \; . \; . \; (9)$$

Thus the shear strain is less than above ; i.e., though using less material, the shear bracing is stiffer without the posts. (Just as a spring with a few coils is stiffer than a spring with many coils.)

With double bracing of this type, Fig. 7 (c), the values of "g" and stress are halved.



Figs. 7

If the chords are flexible, failure may occur by premature buckling over the chord's length between battens. This local strut stress provides the critical value for the stress under load obtained from equation (4). The ratio of the local effective length to the distance between nodes may be all-important. With the lattice braced column, as used in structural steelwork, the problem is automatically solved by the sturdy proportions of the design. Transmission towers, etc., are another matter ; the author can only refer the reader to ref. (4).

Lattice Braced Columns with Unequal Chords

There will be no change in the strain due to unit shear force. The values of "g" are independent of the area of the chords.

The neutral axis of the column will move from the centre towards the larger chord. This affects the value of the eccentricity.

Batten Braced Columns with Equal Chords

It has been shown, ref. (2), page 145, that

$$g = \frac{ab}{12EI_B} + \frac{1.2a}{bA_BG} + \frac{a^2}{24EI_C} \times \frac{1}{1-\alpha} \quad . \; . \; (10)$$

where the terms, see Fig. 8, are respectively due to the local bending of the batten, the shear $\left(\frac{a}{b}Q\right)$ in the batten (assumed of rectangular cross section) and the local bending of the chords. The expression $\frac{1}{1-\alpha}$ gives the influence of the local end load on the bending of the chords ; "α" being the ratio of the local chord load to the Euler load of the chord, taking—for very stiff battens—the batten spacing as the effective length, i.e.,

$$\alpha = \frac{mP}{\dfrac{\pi^2EI_c}{a^2}} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; (10a)$$

Tests justify equation (10) if there are at least six panels, see ref. 2, page (147). Unfortunately the ranges of stiffnesses of battens and chords are not given.

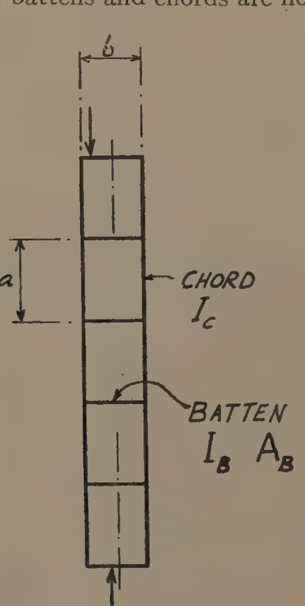


Fig. 8

For a centrally loaded column, as treated in ref. (2), $m = 0.5$. For equally eccentrical loading, mP will vary along the column, being $\frac{1}{2}A$ times the stress from equation (4). As an approximation for "m," take the value at the column ends, $\left(\frac{P}{2} + \frac{Pe}{b}\right)$; this, however, will not be conservative.

The complementary shear, $\frac{aQ}{b}$ (which provides the increment of end load in the column) gives local bending in the batten. The maximum local bending stress in the batten which occurs at the panel joint, is

$$= \frac{\frac{a}{b}Q \times \frac{b}{2}}{Z_b} = \frac{aQ}{2Z_b} \quad . \; . \; . \; . \; . \; . \; (10b)$$

$$\text{The shear stress in the batten} = \frac{1.5\,aQ}{bA_b} \quad . \; . \; . \; . \; (10c)$$

The shear force is taken up the panel in the individual chords. The resulting local bending stress is a maximum at the joint.

$$= \frac{\frac{1}{2}Q \times \frac{a}{2}}{Z_c} = \frac{aQ}{4Z_c} \quad . \; . \; . \; . \; . \; (10d)$$

Thus bending stress is additive, at the extreme fibre, to the uniform stress in the chord given by equation (4).

The individual chord member can now be considered for known stresses which vary along the length of the column. The equivalent length of the chord member is unknown. The author does not claim to have anything useful to suggest as an approximation to the equivalent length, save in the two limiting cases. When the battens are extremely flexible compared with the chords, the equivalent length of the chords approaches the overall length of the column—and the extremely flexible battens are serving no useful purpose since the chords function separately. When the battens are very stiff comparatively, the equivalent length of the chord is of the same order as the spacing between battens. In structural steelwork, the empirical rules for battens tend to provide stiff battens.

Batten Braced Columns with Unequal Chords

Examples of unequal chords are often found in buildings with an overhead travelling crane.

An assumption is made that the shear stiffness of a panel can, when divided by the batten spacing, express the shear stiffness per unit length of the column.

The coefficient "k," giving the proportion of shear kQ in the left chord, can also be used for approximate calculation of Vierendeel girders. It gives both the division of shear between the chords and the position of the point of inflexion of the battens, and equation (11) agrees with the experimental determination of the point of inflexion of ref. (5).

The element in Fig. (9) is acted upon by the shear $\pm Q$ horizontally and $\pm \frac{a}{b} Q$ vertically. The coefficient "k," is obtained from the conditions of continuity. The element $ABCD$ can only join a similar element above or below, when

$$AB = CD$$

$$b - (d_1 + d_2) + (d_3 + d_4) = b + (d_1 + d_2) - (d_3 + d_4)$$

$$d_1 + d_2 = d_3 + d_4 \quad . \; . \; . \; . \; . \; . \; . \; . \; (a$$

The batten is acted on by clockwise moments of $(1-k)Qa$ on the right end and of kQa on the left. The angular deformations at the batten ends are

$$\theta_L = \frac{kQab}{3EI_B} - \frac{(1-k)Qab}{6EI_B}$$

$$= \frac{Qab}{6EI_B}(3k-1)$$

$$\text{and } \theta_R = -\frac{kQab}{6EI_B} + \frac{(1-k)Qab}{3EI_B}$$

$$= \frac{Qab}{6EI_B}(2-3k)$$

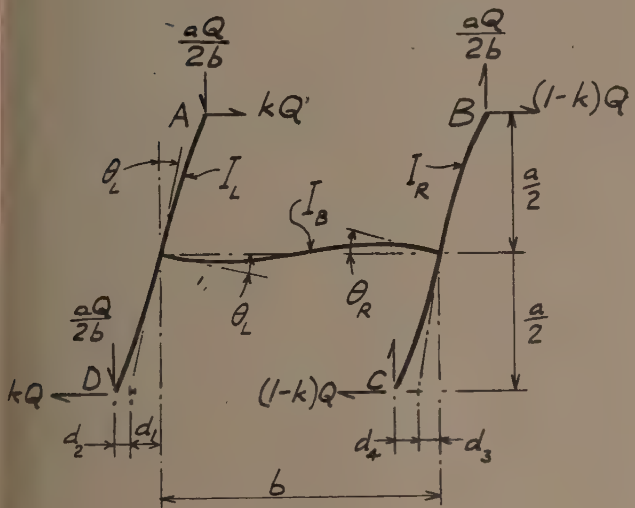


Fig. 9

The chord deflections due to the bending of the batten are

$$d_1 = \theta_L \times \frac{a}{2}$$

$$d_3 = \theta_R \times \frac{a}{2}$$

From the case of a cantilever with a concentrated load at the tip,

$$d_2 = kQ\left(\frac{a}{2}\right)^3 \frac{1}{3EI_L}$$

$$= k\frac{Qa^3}{24EI_L}$$

$$d_4 = (1-k)\frac{Qa^3}{24EI_R}$$

Substituting for "d_1," etc., in (a) and simplifying, the coefficient required is

$$k = \frac{6\dfrac{b}{I_B} + \dfrac{a}{I_R}}{12\dfrac{b}{I_B} + \dfrac{a}{I_R} + \dfrac{a}{I_L}} \quad . \; . \; . \; . \; . \; . \quad (11)$$

The strain of the element due to unit shear force is

$$g = \frac{1}{Q} \quad \frac{d_1 + d_2}{\dfrac{a}{2}}$$

$$= \frac{ab}{6EI_B}(3k-1) + \frac{a^2}{12EI_L} \times k$$

Substituting for "k"

$$g = \frac{a}{12E} \quad \frac{\dfrac{12b^2}{I_B{}^2} + \dfrac{4b}{I_B}\left(\dfrac{a}{I_R} + \dfrac{a}{I_L}\right) + \dfrac{a}{I_R} \cdot \dfrac{a}{I_L}}{\dfrac{12b}{I_B} + \dfrac{a}{I_R} + \dfrac{a}{I_L}} \quad (12)$$

To obtain the same shear flexibility for the half element at the end of the column, Fig. 10, we require,

$$d_5 + d_6 = d_1 + d_2$$

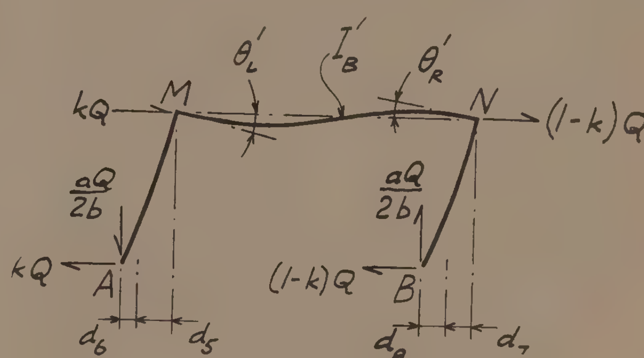


Fig. 10

Here, d_6 is identical to d_2, being the deflection of the same cantilever under the same load. The moments applied to the batten are $\frac{1}{2}(1-k)Qa$ at the right end and $\frac{1}{2}kQa$ at the left. As above.

$$\theta'_L = \frac{Qab}{12EI'_B}(3k-1)$$

$$\text{and } d_5 = \theta'_L \times \frac{a}{2}$$

Thus d_1 and d_5 are equal if

$$I'_B = \tfrac{1}{2}I_B \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (12a)$$

This is the condition to maintain the same value of "g" at the column ends. Since it is normal practice to use a heavier end batten, it will be conservative to treat the end half panel as having the same shear stiffness as the rest

References

[1]D. H. Young. Shearing Stresses in Steel Columns. The International Association of Bridge and Structural Engineering. 1933-4.

[2]S. Timoshenko. Theory of Elastic Stability. First edition, 1936.

[3]E. H. Salmon. Materials and Structures, Vol. 1, p. 284.

[4]H. W. B. Gardner and W. H. Gomm. Tests of Transmission Line Towers. THE STRUCTURAL ENGINEER, Jan. 1950.

[5]Louis Baes. Travaux, Sept. 1937, in French.

[6]John Case. The Strength of Materials, third edition, 1938.

[7]S. Timoshenko. Strength of Materials, Part I, second edition, 1940.

The Structural Engineer as Arbitrator, Expert Witness and Advocate

Discussion on Mr. G. B. R. Pimm's Paper*

The Author's Opening Remarks

In presenting the paper, the author said that in arbitrations relating to building contracts the issues might be wholly legal, wholly technical, or partly legal and partly technical. With regard to the first, he was doubtful whether the dispute ought to be referred to arbitration at all. It could probably be settled much better in the Courts. If it were referred to arbitration the arbitrator should be a lawyer. It was equally certain that when the matters in dispute were wholly technical, the arbitrator should be a technical man, and it was of the utmost importance that he should be appointed as soon as possible after the dispute arose. This was specially important when any kind of structural failure or defect was involved, for the arbitrator could then form his own opinion, from actual inspection, and examination of the facts, and would often be able to reach his decision without further evidence.

Such considerations also resolved the question as to whether the hearing should be formal or informal. Although it was always assumed that it should be formal, there was nothing in the Act to that effect. The Act said a great many things about the appointment of the arbitrator, what would happen to him if he misconducted himself, and other matters, but gave little or no guidance as to the actual hearing. And, after all, that was in line with the law relating to things in general. In every human activity, whether it is driving a motor-car, going fishing, or getting married, we are subject to the law relating to those matters, but this does not mean that our every action should be determined by legal requirements. Furthermore, there is this great difference between such things and arbitration—that if we broke the law in those things, the custodians of the law could step in and call us to account, but in arbitration the law would not intervene unless asked to do so by one of the parties.

Nevertheless, the author hoped he would not be understood to mean that in excluding the law so far as possible, the lawyers should also be excluded. There can be no doubt that in the presentation even of purely technical arguments, they are often invaluable. He would only urge them not to introduce the law in cases where both parties would be content that the issue should be decided on the facts, and on normal practice.

Coming to expert evidence, the author said the only matter to which he wished to refer was the rather prevalent idea that the expert witness cannot do his duty to his client without departing from the truth. That of course was a libel. Nevertheless, the oath to speak "the whole truth" did involve certain ethical principles. Obviously, it only meant the truth in so far as the witness knows it, but this does not quite settle the point. Engineers, probably better than most people, were aware that nobody knows the whole truth about anything. There was always some item of the truth, not already known, which could be ascertained by the expenditure of further time and money—by research for example. What was the position of the expert witness in that connection? Everyone would agree that he should expend that time, and recommend the expenditure of that money, within reason, if it would help his client's case, but was it still his duty to do so if he knew that the information so obtained would be adverse? The author suggested that since it would be obviously impossible to persuade a client to incur expense in order to procure his own downfall, the expert in his evidence could only emphasise that he spoke only out of his own knowledge of the truth.

Mr. Pimm said he was particularly glad to see that they had with them Mr. A. H. Ley and Mr. H. Q. A. Reeves. It had been his privilege to be concerned in an Enquiry in which Mr. Ley had handled the applicant's case very brilliantly, in the capacity of advocate, with Mr. Reeves as the principal expert witness, and he hoped they would both take part in the discussion.

Proposing a vote of thanks to the lecturer, the Chairman said he thought Mr. Pimm in his paper had given them an excellent example of brevity.

There were two points in the paper on which he would like to comment.

The first was the question of expert witness. He strongly advised any of them, provided they had the qualifications, not to shirk that task, for he knew of nothing more calculated to make them think clearly when endeavouring to prepare a draft of the evidence.

The only criticism he would make of Mr. Pimm's paper and his presentation of it was that he made it appear too easy. That was just the natural outcome of his own facility in the matter.

Mr. Douglas Weare (Member), opening the discussion, said that he had been consulted recently by a firm of solicitors to report on a collapsed structure and to advise their client upon the adequacy of the design and the cause of failure.

From a personal discussion with the client before visiting the site, he formed the opinion that the design was unsatisfactory, and a careful site examination of the collapsed structure completely confirmed that opinion. At the conclusion of that investigation Mr. Weare found himself in the position that he would have been far better employed on the other side.

The situation was one in which a consultant acting as an expert witness found himself due to receive his fees for producing a report which would utterly destroy his client's case and possibly render him liable to heavy damages; the exact opposite of what he was employed to do.

In this case Mr. Weare offered to withdraw without producing a report. This offer was not accepted by the

**Read before the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1, on Thursday, November 22nd, 1951; published in* THE STRUCTURAL ENGINEER, *Vol.* XXIX, *No.* 11, *pp.* 289-293. (*Nov.*, 1951.)

[cl]ient's solicitor, however, and the report was submitted. [T]he case was, fortunately, settled out of court.

Changing sides being out of the question, Mr. Weare [a]sked Mr. Gower Pimm for his views, in the circum[s]tances, of the offer to withdraw without submitting a [r]eport. Was such a proposition ethically correct ?

Also, in the event of such a case going to Court and [t]he other side becoming aware that a defendant had [e]ngaged an expert witness whose evidence could only [b]e damaging to the defendant's case, could not the other [s]ide subpœna the defendant's expert witness and use [h]is evidence against his own client ? In those circum[s]tances how should such an unfortunate witness conduct [h]imself ?

In other words, in such a case, how should the balance [b]e adjusted between the ethics of loyalty to one's client [a]nd the ethics of justice to the community ? Was it right [t]o suppress honest evidence simply to win a case and [s]hould an expert witness consider himself bound by [l]oyalty to his client, right or wrong, to the exclusion of [a]ll other considerations ?

Mr. G. W. Tookey, Q.C., remarked that since guests [h]ad been invited to take part in the discussion, and as a [g]uest coming from a different profession, the legal [p]rofession, he would like to say a few words on Mr. Pimm's paper. First, he would like to congratulate [h]im on the paper because it referred to many matters [o]f interest which were perhaps not generally appreciated.

One of the points which Mr. Pimm had quite rightly [e]mphasised was that arbitration began to lose its attrac[t]ion unless the proceedings were conducted with con[f]idence in the arbitrator and in a spirit of co-operation [b]etween the parties in an endeavour to get a settlement [o]f their dispute. Unless those conditions were present then arbitration was not really suitable and one would be far better off in the courts.

To put it another way, one might say that if there was a technical dispute that needed to be settled then one might go to arbitration. That was why they always put arbitration clauses in engineering contracts. But if the dispute was in the nature of a quarrel, or purely a legal dispute, then the Law Courts provided a far better forum. At any rate, he thought no one could complain that they did not provide in this country adequate facilities for the settlement of disputes. From the various forms of arbitration before lay arbitrators, professional arbitrators, official referees and trials in court, there was no difficulty in choosing a tribunal of any desired calibre for the determination of the matters in question.

Arbitration could be of the simplest character, or one could make it as formal and as elaborate as one liked. Perhaps one of the simplest forms of arbitration occurred when two motorists happened to be involved in a collision at some cross-roads and their cars suffered superficial damage. The motorists got out and started an argument as to who was to blame, and eventually getting nowhere, one of them looked up and saw a face at a window, and said, " Let's ask him about it." They shouted, and asked the face at the window if he saw the accident. He replied that he had, and one of the motorists asked, " Who was to blame ? " and the face replied " You were." The decision was accepted, damages were agreed at 30s. and paid on the spot, and everyone was perfectly satisfied !

As regards expert evidence, Mr. Pimm had, for the first time as far as he knew, put down what were some very useful practical points on giving expert evidence, and more particularly, and it was there that he was interested, on the relationship between expert and counsel. As regards examination and cross-examination there had to be the closest understanding between expert and counsel ; in highly technical cases they could not get on without it.

As far as cross-examination was concerned, Mr. Pimm had rightly referred to the assistance which the expert could give to counsel in the cross-examination of the expert on the other side. That was perfectly true, and there was only one thing, which perhaps the lay client was more apt to forget than the expert, and that was the fact that the cross-examination must be built up on definite lines. The starting-point for any cross-examination had to be an agreement between counsel and the witness on certain fundamental things. Having reached agreement up to a certain stage, counsel then proceeded to see how far the expert would go with him. He would not say it was impossible, but it was very difficult to change the line of cross-examination and go back and alter the premises. In practice it was very difficult to slip in an additional premise in the course of cross-examination. He knew if was often said, and it had been often said to him (when he pointed out that a question involved a fact that they had not so far canvassed), that the witness would accept the new fact, but the witness usually would not accept at short notice a new premise.

Where the expert sitting in front of counsel could be so valuable was if he at the right moment could assist counsel by suggesting useful and pertinent questions to carry on a line of cross-examination which was based on the original premise and did not break the continuity.

There were one or two other points which Mr. Pimm had suggested he might deal with. He had very pleasant recollections of a recent case in which Mr. Pimm and he had both been engaged. It was a titanic struggle which went on for a long time. They were trying to find out why a dock wall fell down, and then they tried to tackle the question as to why it ever stood up, and then having regard to the fact that it had stood up for a hundred years, why it should want to fall down.

In the course of that long case the lawyers became very knowledgeable on the question of soil mechanics in relation to London blue clay, but they never found an answer to their question. That was not the fault of the experts who assisted counsel and the court in that case. Mr. Wentworth-Sheilds was on the other side, and the experts' assistance was in the best traditions of the profession and in accordance with the precepts which Mr. Pimm had so wisely, if he might say so, put forward in his paper.

He would conclude by answering his question of what was " the truth, the whole truth and nothing but the truth." He had expressed a difficulty. Mr. Pimm was not alone in that, he was in good company in expressing that difficulty. There was once a famous murderer who felt the same difficulty. He chose to give evidence in his own defence and when it came to taking the Bible in his hand and repeating the oath, he said, " How can I tell the whole truth ; I don't know it."

The answer to Mr. Pimm was exactly the same answer as was given to the murderer : " You are only required to tell the truth in as far as you know it."

Mr. Rosenvinge, a visitor, said he felt sure that many of them before hearing Mr. Pimm's paper, were under the impression that arbitration must always involve members of the legal profession acting for and quite often representing parties at the hearing. Mr. Pimm

had explained to them how arbitration cases concerned entirely with engineering matters could be conducted and settled by engineers without recourse to a battle in which legal representatives might choose to make an issue of matters having relatively little bearing on the main dispute. Of course, as Mr. Pimm had said, all cases could not be settled solely in the light of the arbitrator's experience, his administrative technical knowledge and sense of equity. It might be that on occasions a point of law did arise, but surely in such instances counsel could be consulted in the same way as an engineer was called upon to give evidence as an expert witness in a court of law. That solution appeared just too easy, but it would be quite an advantage from the engineer's point of view on account of the costs which often resulted from legal proceedings. Many consultants and contractors, when disputes arose between them over a contract, often agreed between themselves to accept a compromise settlement, and of course a compromise was not always very satisfactory. It was quite clear, he thought, that arbitration could be conducted on relatively informal lines and if an arbitrator's fees and expenses could be estimated in advance that method of settling the disputes which frequently arose on all public works contracts would surely become very much more general.

Mr. A. H. Ley (Member of Council) said a reference had been made to a recent case in which Mr. Pimm was concerned. In dealing with that particular case he had learned the lesson that the preparation for an arbitration called for just as much quiet thinking as the preparation of a proof by an expert witness. The second thing he had learned was that if one was called on to deal with arbitration and to conduct it without the benefit of legal assistance, or, to put it another way, without it being conducted by a member of the legal profession, the first thing one had to do was to be very careful in the choice of one's professional witnesses. Arbitration could only be really successful if the arbitrator was someone in whom all parties had complete confidence. Where the arbitrator was a man whom it was known understood precisely what was being talked about, then he thought the structural engineer was in an advantageous position over the lawyer in presenting his case, and in the particular matter referred to by Mr. Pimm they had that great benefit. That case was not a matter of law at all. It was a matter of fact, and, while it took quite a long time to prepare that case, they had been able to dispose of it in a morning and half an afternoon. He could conceive that it could have gone on for two days, or perhaps a week, in the normal courts of law. They finished the hearing and were able to part as good friends with the party on the other side, although at one time it did look as though what had started as a dispute was going to develop into a quarrel.

The point about confidence was, he felt, the basis of success in arbitration.

Reference had been made in the paper and again that evening to the London Court of Arbitration. For many years that court was presided over by Lord Leverhulme, at that time the Chairman of that great concern which bore part of his name. That company on many occasions every year had to have recourse to arbitration and frequently its cases were referred to the London Court of Arbitration, and on no occasion whatever was there an objection raised by the other party to that course being adopted. It was because of this perfect confidence in the arbitrator.

Dr. E. H. Bateman (Member) said that he had listened with great interest but there were one or two points which he would like the author to explain. What, actually, was the difference between a technical point and a legal point? Could that be made clear by a simple case? Was there, for example, a clear distinction between a case which went to arbitration or to the Court of Chancery or the Queen's Bench?

He said that he had heard of a case of an expert witness who had changed sides, not for the very good reason which had been mentioned in the discussion, but because he found that one of the parties had a longer pocket than the other! He asked the author to comment on the ethical standards of the expert witness.

Finally, as a matter of general interest: the layman had an idea of legal costs as being tremendous. What was the actual ratio on the average? Could one get arbitration for about 1/10th of the cost of legal proceedings, or was it like the ratio of the case which had been put up, a relation of 30s. as against £300?

Mr. H. Q. A. Reeves, a visitor, said that it was often that the technical expert was called into a case rather late in the day. He might find that his client's case was ill-founded. Whatever he found, it was his duty to stand by his client, once he had accepted instructions and was retained, provided that he was satisfied that there was nothing illegal contemplated.

Technical experts should insist on seeing all the papers in the case, before accepting instructions.

He had found on occasions that Counsel's "Advice on Evidence" did not meet the particular case, because incorrect technical information had been given to Counsel. It was sometimes necessary for the technical expert to re-write the case in order to show it in its true light.

If, after a technical investigation, the client did approve of the evidence you were prepared to give in court, they need not call you. Alternatively, they could terminate your services at any time by letter and a request for your account. Mr. Reeves did not know whether or not the opposition being aware of such a termination, could subpœna the expert with a view to making him disclose his findings in court.

He imagined that this action could be taken, but he would like advice on this point.

Mr. W. R. Howard (Member) said since Mr. Pimm had been invited to arbitrate on questions of ethics, could he put a problem to him?

Some months ago he had been approached to act as arbitrator, and he had foolishly consented to do so. A few weeks afterwards the firm of solicitors approached him again and asid, "You agreed to act as an arbitrator on behalf of our clients. Now the other side have appointed an arbitrator, and the first job you two arbitrators have to do is to get together and appoint an umpire." This has been done.

Mr. Howard would like to know if at the hearing of the case he was expected to be partial or impartial!

Mr. Leslie Turner (Past-President) thought the Institution was to be congratulated on having this paper in its proceedings because it did really crystallise so much information that it was difficult to find in and assimilate from books. One usually had to come by it through hard experience.

He was very glad to hear from high authority present that truth was not really divisible. When uttering the

th for the first time one might be pardoned for ondering if "the truth, the whole truth and nothing t the truth" referred to various shades of truth; and was reassuring to know that it was really a time-noured tautology to extract "truth" as far as such d been revealed to the witness.

He thought that an engineer's best approach to legal arbitration proceedings was by way of an apprentice-ip as technical witness. He would then learn much watching and listening, apart from his own particular deal of giving evidence. He would then be better epared if the cloak of arbitrator, and certainly that of vocate, descended upon him. Again, recourse to is paper would be enormously valuable.

From his own experience there was one matter he ould like to have resolved if possible. When accepting pointment as arbitrator one did not know until after ceptance whether the major point in dispute was chnical or legal. It was of course similar with the ourts. Now, an engineer preferred and might justifi-ly consider he was better qualified to deal with a chnical issue rather than a legal one, despite the fact at his path in this respect was excellently indicated the author. The reverse might be thought to hold r the Courts where judge and counsel were often called pon to wrestle with deep technical details that had ken engineers years to understand fully.

Several times he had been appointed where the spute was of a purely legal or contractual nature, hile on other occasions he had laboured to get over chnical evidence in court cases where the issue was, or ould (in his opinion), have been predominantly tech-ical and better dealt with by a technical arbitrator.

Could some preliminary sieve be devised whereby ses were sifted out to some extent, and the relevant lteration be made in contract documents?

Lt.-Colonel G. W. Kirkland, M.B.E. (Member of ouncil) pointed out that a number of forms of contract ow included an arbitration clause, and the arbitrator as usually a person to be appointed by the President f one of the Institutions connected with the type of ork undertaken.

During the war he had had the job of handling many Var Department contracts. It had always occurred o him as remarkable that in a works or construction ontract between H.M. Government and another party, Government official was invariably referred to as rbitrator and appointed as arbitrator in the contract ocument.

That had struck him as being unethical, and he would ppreciate Mr. Pimm's opinion.

Mr. G. Polsen said that when reading the article and ubsequently when listening to the discussion, he felt hat Mr. Pimm thought that the law should at no time nterfere with the affairs of arbitration. He thought e would agree that that summarised his feeling that an through the article. He sympathised with that as a eeling because if people were going to decide on arbitra-ion as a means of settling their dispute why not leave hem completely free to abide by the decision of the rbitrator and cut the law out altogether? That in the arly days was a long-established part of our institutions nd at one time it was completely free. He thought he vas right in saying that about the middle of the 17th century the law became somewhat jealous of their urisdiction and almost clamped down on arbitration nd decided that in many cases it would exercise its jurisdiction for settling disputes in that way. To-day, he thought it was true to say that the Courts took the view that people ought to be free to arbitrate without any interference from the Law Courts. In fact, in a recent case it had been made clear that apart from abuse in the administration of arbitration, the Courts would not interfere with the award of an arbitrator. But he thought it was going too far to say, as Mr. Pimm did, that the impact of the law on arbitration should be no more effective than the impact of the law on fishing or motoring or other such things. But he thought the comparison broke down because the arbitrator was acting on matters which were at issue between parties, but there were, notwithstanding, facts which were at issue and the arbitrator was dealing through his decision on them in the capacity of a judge, and in such a case he thought it right that the Court should have at some level the right to look into and inspect the proceedings to see that they had been conducted according to the law of natural justice.

With regard to the engineer as his own advocate he had serious views and was not making his contribution because he had a vested interest in the legal profession. But it was said, and he was sure lawyers fully agreed with it, that there was no fool like his own lawyer. The reason being that he was too near the facts and too much involved to see the issue judicially. There was also the view that the legal necessities could be easily overdone and so easily obscure the principles of equity. He would suggest that it was the lawyer's view that there was an equal danger in such proceedings of the technical necessities being overdone and thus obscuring the principles of equity, and it was because he felt that, that he felt some doubt about the general principle of the engineer being his own advocate. He thought it was too near the trees to see the wood. He was too involved in the intricacies of the thing to be able to present the thing with the clarity which was required. They needed those balanced views of things.

With regard to expert evidence, he could only say that to read of an expert who could so judge what Counsel was likely to require as to have typed sheets prepared for him was a luxury from Counsel's point of view. He congratulated Mr. Pimm on having arrived at such a stage in being an expert witness, and could only hope that in any proceedings in which he might be involved he could have such an expert witness.

Mr. Pimm, replying to the discussion and the questions, thanked the members for the very kind way in which they had received his paper.

Mr. Douglas Weare had raised a point which he believed he had covered in his opening remarks. If Mr. Weare's very proper offer to withdraw was not accepted, and he had been subsequently called, it was obvious that no blame could attach to him if his evidence was damaging.

He thanked Mr. Tookey most sincerely for his remarks. There was nothing he could possibly add to them.

Mr. Rosenvinge had made a suggestion that counsel should be called in evidence in the same way as engineers were called. That was provided for, in a way, in that it was quite common for an arbitrator to have a legal assessor to sit with him. He could then consult him on points as they arose, and it certainly did dispose of a good many legal points.

Dr. Bateman had asked what was the difference between a legal and a technical case. That could be only decided on the facts, but in a general way a dispute

as to whether something had been properly designed or not would be definitely a technical issue ; the construction of a particular document might be a legal issue. He was afraid he could not possibly answer the other question as to whether a thing would go to the Court of Chancery or to the King's Bench.

His question as to cost was one that could only be decided on the actual circumstances. An arbitration in which only technical men were engaged, and which was confined to technical issues, would certainly be much less costly than an action in the Courts. On the other hand, if legal gentlemen were engaged, the Courts would have the first call upon their time, and therefore, much as they might regret it, they would not be able to make their attendance at an arbitration as consecutive as they would wish in the event of there being several adjournments of the hearing. In other words, if they had a purely legal arbitration, that arbitration would very often have to wait on the convenience of the Court, and might drag on very much longer than it would in the Court, where it would normally go on from day to day and be finished out of hand.

He also thanked Mr. Reeves for his remarks, and the answer which he gave to Dr. Bateman.

Mr. Howard asked a question about the arbitrator. That was provided for in the Act, and when two arbitrators disagreed at the outset they then appointed an umpire. If they could agree on that, they then retired and he took over. The general tendency now was to discourage the practice of appointing two arbitrators.

Mr. Turner had raised a point which had interested him and might interest the Institution. He had suggested that the disputes should be sorted out as between legal and technical. It really meant that the form of arbitration clause in the contract to which they were accustomed might with advantage be altered. He had wondered if it was not possible to frame a clause which would ensure that a dispute would go to the Court or to arbitration according to the kind of dispute it was.

In reply to Col. Kirkland, Mr. Pimm said he had had some experience of enquiries, which were not arbitrations within the meaning of the Act, where the arbitrator was a permanent official appointed by the Department concerned, to preside at such enquiries. In such cases, as would be expected, the proceedings were conducted with the utmost fairness. The case mentioned by Mr. Ley was a case in point, and they certainly had no reason for complaint there. In ordinary building arbitrations it was a bad practice. As he had pointed out in the paper it was discouraged by the Act, which gave the parties the right to apply for permission to revoke the authority of the arbitrator, even where they knew, when agreeing to his appointment, that he might not be impartial.

Mr. Polsen, he thought, must have misunderstood him in two matters. He did not suggest that an engineer should be advocate in his own case, but merely that in a purely technical case an engineer should be engaged as advocate for one side, and another engineer as advocate for the other. That, he thought, cleared the point, but it would be only in cases which were clearly from beginning to end of a technical nature.

He thought the difficulty about the impact of law on arbitration was really a question of case law. The case law of arbitration must obviously always be very incomplete because only those arbitrations which had been transmitted to the Court, for one reason or another, were reported. Therefore the body of case law relating to arbitration could not be complete. At an Arbitration not long ago it was suggested that the duty of the quantity surveyor was simply a geometric one of measuring quantities. That might have been true, say, 30 to 50 years ago, but it was not true now, and in technical matters generally, by reason of the constant changes in practice not recorded in case law, disputes could only be settled by the arbitrator's and the advocates' up-to-date knowledge of custom and practice in the industry. That was more to be desired than references to case law, which might be hopelessly out of date.

As to excluding the law, he thought it might be illustrated by a story. A friend of his was extremely bad at getting up in the morning. He tried all sorts of alarm clocks without success. So in the end he had a cupboard made in his bedroom with an open grille front. He procured an alarm clock of the type that went on and on and did not stop. He wound it up, set it, and put it in the cupboard, locked the cupboard and took the key downstairs. He had not relinquished his right to sleep on in the morning, but he had made it so difficult that it was extremely unlikely that he would do so.

Mr. Pimm concluded by thanking those present for the kind manner in which they had listened to what he had to say.

The Chairman, closing the meeting, thanked Mr. Pimm warmly for the way in which he had answered the many questions.

This concluded the meeting.

Further Written Reply by the Author

The author is conscious that he dealt very inadequately with a group of questions raised by Mr. Weare and Mr. Reeves. An engineer who has been retained to report, and subsequently to give evidence, finds that the evidence he would give would be damaging, and reports accordingly. Then one of three things happens. It may be decided (a) to call the witness and chance it, (b) to continue to retain him but not to call him, or (c) to terminate his services. The question is, whether in either of those cases the other side can call him under subpœna, and if so, what should the witness do? The answer to the first part of the question, fortunately for the witness, is for the lawyers, and not for him, to decide. If he is so called he must of course attend, and all that he can do is to answer truthfully the questions put to him. If his replies are damaging it is no fault of his. The author would imagine that the situation is hardly likely to arise except in the third case, i.e., if the client has terminated his services, and even then, as in the other cases, he will have powerful protection on the grounds of privilege. It would not be permissible, for example, for the opposing counsel to ask him : "Did you submit a report to the defendant, and was it so unfavourable that he decided that it would be safer to have nothing more to do with you?"

From the defendant's point of view probably the safest course would be to call the witness, but ask him only innocuous questions, and to obtain another witness if possible, who feels that he can offer more favourable evidence on the vital points. That would spike the opposition's guns, since whatever right they may have by way of subpœna, if the witness is not called by the defendant, they could hardly have that right if he is so called.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, March 27th, 1952, at 5.55 p.m. Mr. Walter C Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership ?

Students

ALLEN, Derek, of Harrow, Middlesex.
BELL, James, of Manchester.
BRADSHAW, John Richard, of West Kirby, Cheshire.
DAMPIER, Bernard William, of Manchester.
DIAS, Anthony Francis Joseph, of Brighton.
HARVEY, Michael James, of Coventry.
JARVIE, George Sinclair, of Cardiff.
RATCLIFFE, Colin John, of Wirral, Cheshire.
TURNER, John Edward, of Derby.
WARDEN, John Charles, of Hornchurch, Essex.

Graduates

AMOS, Keith, of Nottingham.
BARBER, Hubert, of Eccles, Lancs.
BIRCH, Norman, of Salford, Lancs.
CHOPRA, Ajudhya Nath, B.E.(Civil) Calcutta, of Howrah, India.
CLARKE, Stanley, of Liverpool.
CROSTHWAITE, Donald Rothery, B.A. Cambridge, of St. Albans, Herts.
CURRIE, Bernard Alan, of Andover, Hants.
DOVE, Eric David, B.Sc.(Eng.) London, of Bradford, Yorks.
FARNABY, John Eric, of London.
HELLIS, John Blackwell, of Harrow, Middlesex.
INGHAM, Bryan Ronald, B.Sc.(Tech.) Hons. Manchester, of Cheadle, Cheshire.
KAY, Anthony John, of London.
MATTHEWS, Bertram Lyle, B.Sc.(Civil) Cape Town, of Salisbury, Southern Rhodesia.
MENZIES, Ian William, B.Sc.(Eng.) Glasgow, of Liverpool.
SPINKS, Maurice Sidney, B.Sc.(Eng.) London, of Shoreham-by-Sea, Sussex.
WADDY, Harry Francis, B.Sc.(Eng.) London, of Bromley, Kent.

Members

SHEARER, Charles John, of Glasgow.
WILSON, Alexander John Cope, of Birkenhead, Cheshire.

Transfers

Students to Graduates

AAGAARD, William Valdemar, of Birmingham.
BETTANY, George Angus, of South Benfleet, Essex.
BROWN, David Henry, of Hazel Grove, Cheshire.
CUSSONS, Stanley Harold, of Stockton-on-Tees, Co. Durham.
DANAHER, Martin Denis, B.E.(Civil) New Zealand, of Wellington, New Zealand.
DAVIDSON, Gerald Keith, of Liverpool.
DUNTHORNE, Gareth John, of London.
HARDCASTLE, Frank, of Salford, Lancs.
HARRIS, David William, of London.
HOLLEY, Michael, of Brighton.
WILLIAMS, John, of London.

Graduates to Associate Members

ADAMS, James Ronald, of Boston, Lincs.
CLOWES, Stanley George, of Manchester.
FITCH, Norman Arthur Stanley, of London.
MATHER, Robert, of Flixton, Lancs.
SARGENT, Albert Reginald, A.M.I.C.E, of Buckhurst Hill, Essex.

Associate-Members to Members

BALL, Norman George Thomas, of Bishopston, Bristol.
GARDNER, Rodney Robert, of Croydon, Surrey.
WADDELL, Albert Victor, of London.
WATSON, Frederick Bernard, of Birmingham.
WHITTEN, Joseph, A.M.I.C.E, of Whitley Bay, Northumberland.

OBITUARY

The Council regret to announce the deaths of Alfred BAILEY, Allan Leslie GRIMSHAW, Norman Fitz HARDING, Richard MITCHELL, William James MULLINGS (Members) ; Thomas William WATKINS (Associate) ; Jacques Herbert HEDGCOCK, Robert Wright HOLMES (Associate-Members).

RESIGNATION

Notification was given that the Council had accepted with regret the resignation of Reginald GODFREY (Associate).

EXAMINATIONS

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on July 15th and 16th, 1952 (Graduateship), and July 17th and 18th (Associate-Membership).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1 :—

Thursday, May 22nd, 1952

Ordinary General Meeting (for the election of members), 5.55 p.m.

Annual General Meeting, 6 p.m.

Thursday, June 26th, 1952

Ordinary General Meeting (for the election of members), 6 p.m.

BENEVOLENT FUND

The Annual General Meeting of the Voting Contributors to the Institution of Structural Engineers' Benevolent Fund will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, May 22nd, 1952, at 6.30 p.m.

STANDARD METHOD OF MEASUREMENT

The Standing Joint Committee for the Standard Method of Measurement of Building Works receive from time to time requests for reconsideration or clarification of specific terms in the Standard Method of Measurement; in addition they have before them the recommendation of the Anglo-American Productivity Team's Report which reads "Consideration should be given to the simplification of the Standard Method of Measurement."

To enable the Committee to give the fullest possible consideration to these matters and to the principles involved, detailed suggestions would be welcomed from those interested.

These should be sent by 31st May next to The Registrar, S.M.M.C., The Royal Institute of Chartered Surveyors, 12, Great George Street, Westminster, London, S.W.1

LONDON GRADUATES' AND STUDENTS' SECTION

The Annual General Meeting of the Section was held at 11, Upper Belgrave Street, London, S.W.1, on March 11th, when the following Honorary Officers were elected :

Chairman : Mr. P. L. Harvey (Graduate).
Vice-Chairman : Mr. E. Easton (Graduate).
Hon. Secretary : Mr. C. A. Brown (Graduate).

A visit to the Directorate of Colonial Survey at Tolworth, Surrey, has been arranged for Saturday, May 17th. The work of the Directorate is concerned with aerial surveys and mapmaking. Those taking part in the visit will meet either at Waterloo Station at 9.30 a.m., or at Tolworth Station at 10.15 a.m.

Hon. Secretary : C. Allen Browne, 43, Coolgardie Avenue, Highams Park, London, E.4.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The Annual Business Meeting will be held at the College of Technology, Manchester, on Thursday, May 15th, at 6.30 p.m., preceded by tea at 5.45 p.m.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 28, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

Hon. Secretary : E. R. Deeley, A.M.I.Struct.E., Arranmoor, Adshead Road, Dudley, Worcs.

GRADUATES' AND STUDENTS' SECTION

A meeting will be held at the James Watt Memorial Institute, Birmingham, at 7 p.m., on Friday, May 30th, when short papers will be given by members of the Section.

Hon. Secretary : M. H. Evans, B.Sc., 42, Church Hill Road, Handsworth, Birmingham, 20.

NORTHERN COUNTIES BRANCH

Hon. Secretary : Ian MacGregor, M.I.Struct.E., 9, Ellison Place, Newcastle-upon-Tyne, 1.

NORTHERN IRELAND BRANCH

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

Hon. Secretary : D. G. Drummond, B.Sc., M.I.Struct.E., A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary : E. W. Howells, A.M.I.Struct.E., c/o Messrs. T. Harding & Sons, Ltd., 10/12, Market Street, Torquay.

WALES AND MONMOUTHSHIRE BRANCH

A joint visit has been arranged with the Midland Counties Branch to the Penmaenmawr Welsh Granite Quarries and Llandudno, on Saturday, June 7th.

Hon. Secretary : E. R. Steward, A.M.I.Struct.E., Edrom, Ashleigh Road, Blackpill, Swansea.

WESTERN COUNTIES BRANCH

Hon. Secretary : C. E. Saunders, M.I.Struct.E., Dunkery, Edward Road, Walton St. Mary, Clevedon, Somerset.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

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Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

CORRIGENDA

"The Structural Engineer," February, 1952. p. 28, Expression (3) should read :

$$r = \left(1 - \frac{t}{20}\sqrt{\frac{s}{a}}\right)\left(1 - \frac{t}{40\sqrt{sa}}\right)^2 \quad . \quad . \quad . \quad (3)$$

p.30, the first line of the caption under Fig. 4 should read :

Fig. 4.—Log r plotted as a function of log $(1—T\sqrt{s})\ (1—\frac{T}{2\sqrt{s}})^2$

the second line of the caption under Fig. 5 should read :

those calculated from $r^{\frac{1}{2}} = \frac{1}{2}(1—T\sqrt{s})\ (1—\frac{T}{2\sqrt{s}})^2$

p.31, the caption under Fig. 6 should read :

Fig. 6.—The family of curves $r^{\frac{1}{2}} = \frac{1}{2}(1—T\sqrt{s})(1—\frac{T}{2\sqrt{s}})^2$

p.33, col.1, line 11. For "badly" read "hardly."

Book Review

Simplified Mechanics and Strength of Materials, by H. Parker. (New York : Wiley, 1951 ; London : Chapman & Hall). 275 pp.-xvi, 8 in. × 5 in. 32s.

This book is one of a now well-known series known as "Simplified." It is not a treatise, but is a sound elementary introduction with the text clearly written and the diagrams extremely clear and well done. The author goes straight to the formulæ required for the solution of the various examples worked out, often after only a few lines of text on general lines. These very brief notes, although useful, cannot possibly fully explain how the formulæ are derived.

Chapter 9 deals with continuous beams and restrained beams in about ten pages, while moving loads occupy only four pages. Numerous tables of the properties of materials and sections are given. The sections are of course the American standard sizes. The loads are in lbs., and the stresses are in accordance with American standards. Only very elementary mathematics are used throughout the book as the aim of the author has clearly been to be true to the title of "Simplified." Useful questions are set at the end of each chapter.

The range of the book is shown by naming the headings of some of the principal chapters.

Mechanics and Strength of Materials, Forces, Stresses and Deformations, Properties of Sections, Shearing Stresses in Beams, Bending Moments, Deflection of Beams, Built-up Beams of Two Materials, Columns, Rivets and Welds, Reinforced Concrete, Retaining Walls and Dams.

The stresses given for steel and concrete are in accord with modern practice, with a varying value for the modular ratio.

This book will probably be popular, especially to those who wish to know how it is done rather than why.

N. T.

Tension Impact Testing

By A. C. Vivian, A.C.G.I., M.I.C.E., M.I.Struct.E.

Synopsis

The paper describes two testing machines. A full-size prototype has already been made of one of these and a demonstration model is also available. A demonstration model only is available for the other.

The first machine pulls a tension bar apart and determines the ultimate tensile strength and ductility properties of the bar under the same rate of impact loading as the Izod test. This machine forms the subject of patent applications in Great Britain and the United States.

The second machine is designed to produce an autographic load-extension diagram of a tensile bar at various speeds of loading, either at slow or at "static" rate of loading.

Accompanying the paper are a number of test results carried out on the tension impact machine and compared with static test results.

The paper concludes with some points for discussion regarding the possible merit from the user's point of view of developing a tension impact test and an autographic tensile test.

Acknowledgement is made to the Anglo-Iranian Oil Company for sponsoring the tests and for permission to publish the results and to the Company's staff at Eakring workshops for the manufacture of the tension impact machine. The conclusions from the tests reported are the Author's own opinions.

Introduction

The purpose of this paper is to introduce to the notice of engineers the possibility that a tension impact test might be a preferable alternative to the standard Izod test for determining the shock properties of steel and other materials.

There has always been something mysterious about the Izod test and although it has been of great value in a large number of cases in differentiating between tough and brittle steels, the most usual way in which a structue fails in practice under shock is under tensile shock and not transversely across a notched section.

The machine described below is able to test under tension shock, standard bars which may be plain or notcned, and experiments already carried out prove that a plain bar tested in tension impact is capable of showing the material to be brittle without the added difficulty of cutting a uniform notch in it. The notch, however, if required on a tension bar, is easy to machine.

When a steel bar is stretched to failure in tension, whether in the normal static test (at usual laboratory or works testing speeds) or at the high speed of a tension impact test, the stretch comprises two parts, one an approximately uniform extension up to the maximum strength of the steel (i.e., the ultimate tensile strength) and the other a necking extension.

It is relevant to observe that this fundamental characteristic of steel, which may be conveniently called effective ductility and which the late Professor Unwin determined beyond all shadow of doubt fifty years ago as an intrinsic property of steels and which is well-known to-day, is ignored in both British and American standard tensile specifications.

The tension impact machine provides the means of measuring this uniform stretch at high speed. At static conditions (ordinary rate of loading in a tension testing machine) and slow speeds, the author suggests that an autographic tensile testing machine is also a desirable piece of equipment for steel testing purposes.

Description of Tension Impact Machine

Fig. 1 shows a front view of the machine. It consists of a very rigid steel frame having an upper vertical bar from which the test piece is suspended. The lower end of the test bar suspends an anvil bar having an anvil

Fig. 1.—Tension impact machine. Ready to release tup

with a Morse taper at the lower end. A weight adjustable in seven steps from 123 lb. to 400 lb. slides easily on the anvil bar and on being released by a handle on the right-hand side (see Fig. 2), falls 2 ft. and makes an air-tight seal with the Morse taper. The falling tup or weight, together with the anvil bar and broken part of test bar, then plunge together into an air cylinder (see Fig. 3). The inside rim of the air cylinder is provided with a U-leather. The residual energy of the weight and anvil is absorbed by compressing the air, and the depth of plunge of the falling tup into the air cylinder is recorded by the pencil and chart (see Fig. 3) and is an exact measure of the residual energy (after breaking the bar) plus losses. Fig. 3A is a drawing of the autograph gear. Alternatively, the falling tup may be brought to

rest by a spring loaded platform on to which the tup falls after stretching and/or breaking the test piece. A model of this is available for demonstration.

Fig. 4 shows the differential winch used to lift the weight to its release position. Safety guards are provided as required and also safety bars to secure the

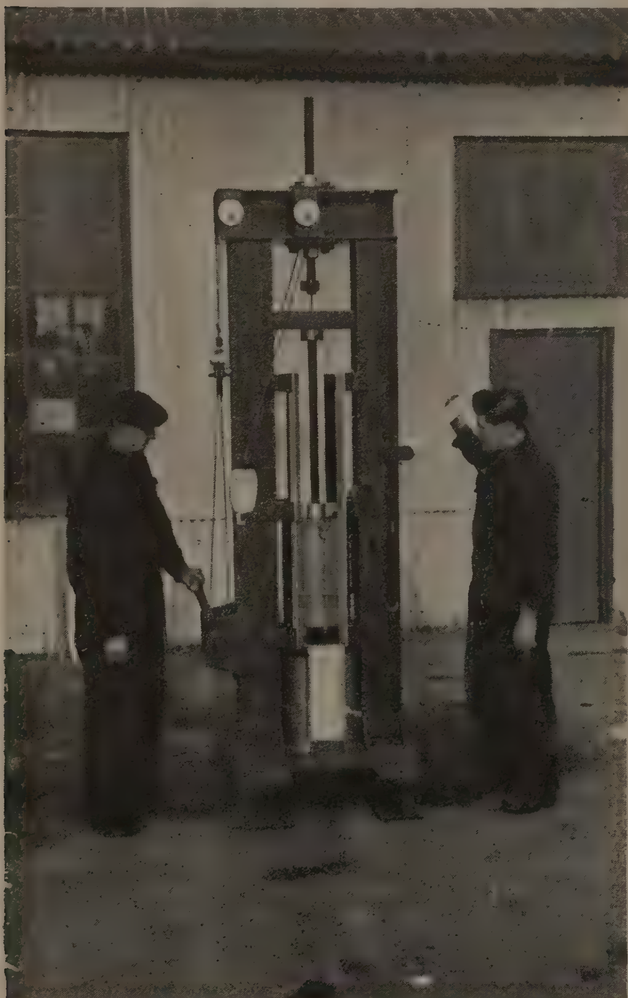

Fig. 2.—Tension impact machine tup released

weight in its starting position, the latter being withdrawn when making a test. The paper chart and pencil record can be read to 1/50 in., and from calibration curves the corresponding energy used in breaking the test bar is read off in ft. lb.

Theory of Machine

Fig. 5 illustrates the theory of the machine.

If W = weight of falling tup,
W_a = weight of anvil assembly.
H = free fall of tup $\doteq$ 2 ft. to correspond with standard Izod machine,
H_c = Combined fall to air cylinder,
H_y = plunge of tup into air cylinder ;
and if E_b = energy to break test bar, ft. lb.
E_f = energy losses due to friction, to making Morse taper joint and to elastic deformation of frame in ft. lb.
E_c = energy to compress air in ft. lb.
and E = total energy,

$$E = W\,H + (W+W_a)\,H_c + (W+W_a)\,H_y \quad (1)$$

and

$$E = E_b+E_f+E_c \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

To calibrate the machine a test bar of known breaking energy is used. This consists of two half-test bars of mild steel joined at the centre with a loop of 16 S.W.G. copper wire. By using small weights over the anvil bar, the breaking energy of this calibration bar was determined to be 11 ft. lb. Alternatively, a cast-iron bar can be used. Such a bar, $\frac{1}{4}$-in. diameter $\times$ 4 in. gauge length, also breaks at between 8 and 12 ft. lb.

Then with each of the seven weights in turn, three or four values of H_y are determined for each weight. Table 1 gives a set of results obtained using the copper wire calibration bar.

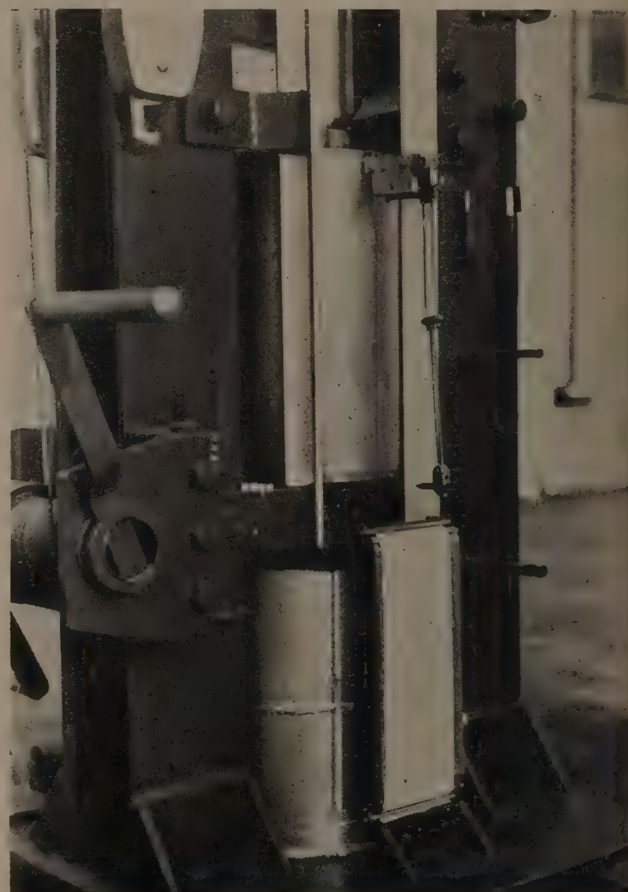

Fig. 3.—Tension impact machine. Air cylinder and plunge recording chart and pencil

TABLE 1.—Calibration Measurements for Tension Impact Machine

Weight No.	Weight of tup and anvil bar (lb.)	Plunge (in.) 1	2	3	4	Mean
1	158.7	4.80	4.80	4.80	4.78	4.80
2	215.2	6.14	6.16	6.18	6.18	6.16
3	259.3	7.00	6.98	6.98	6.96	6.98
4	303.4	7.70	7.68	7.66	7.68	7.68
5	347.5	8.26	8.26	8.28	8.26	8.26
6	391.6	8.76	8.74	8.76	8.74	8.74
7	435.7	9.16	9.16	9.16	9.14	9.16

Date of calibration 13/2/50. Barometer 29.1 in. Hg, Mercury

Then E is calculated from Equation (1) ; and from (2 the value of (E_f+E_c) for each value of H_y is compute and plotted so that the relation between (E_f+E_c) an H_y is determined for any value of H_y.

TENSION IMPACT MACHINE

FIGURE 3A- CALIBRATION FITTING

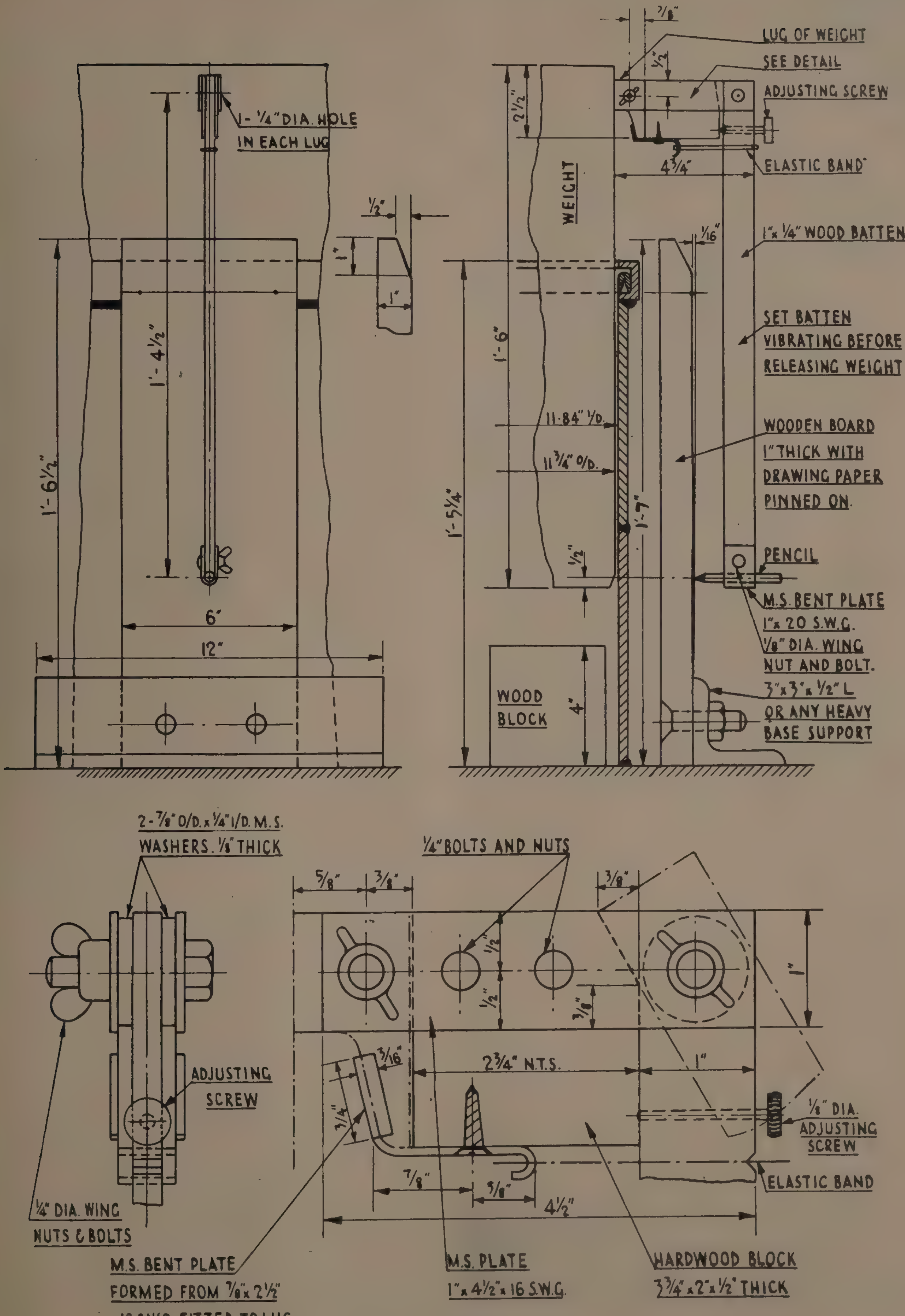

Fig. 3a.—Calibration fitting

Fig. 6 shows the relation between $(E_T + E_c)$ and H_y plotted to base H_y in graphical form for the values in Table 1.

For each of the seven weights the value of E is then calculated from Equation (1) for various values of H_y up to the limit of H_y which the particular weight can reach. Then using Equation (2) :—

$$E_b = E - (E_t + E_c) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

In this equation E is known, and also $(E_t + E_c)$ for each particular value of H_y, so E_b is determined for each

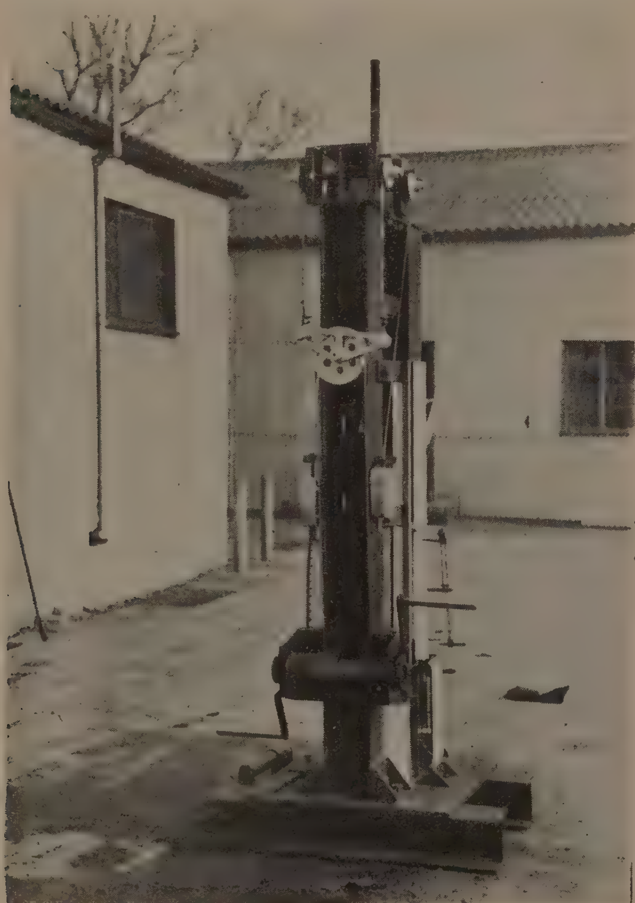

Fig. 4.—Tension impact machine. Differential winch

of seven weights for the relevant values of H_y. Fig. 7 gives the calibration curves so determined for the machine described. The capacity and range of the machine are given in Table 2.

TABLE 2.—Capacity and Range of Prototype Tension Impact Machine

Weight No.	Weight (lb.)	Max. breaking Capacity (ft. lb.)	Range (ft. lb.)
1	123.0	275	50 to 150
2	179.5	400	150 to 300
3	223.6	500	250 to 400
4	267.7	600	350 to 500
5	311.8	700	450 to 600
6	355.9	800	550 to 750
7	400.0	900	700 to 850

In addition, small weights are provided to give a range from ½ ft. lb. to 50 ft. lb., by trial and error (since the residual energy cannot be measured on this particular machine for these small values).

It will have been noted that the method of calibration and preparation of the calibration curves eliminates any error in computing the value of energy to break the test bar, since the losses are the same whether a calibration bar of known breaking energy or a test bar is used, and the losses accordingly cancel out in the computation.

Physical Measurements on Test Bars

The test bar normally used is No. 1 of Fig. 8. The No. 2 test bar is used when material under ⅜-in. thick

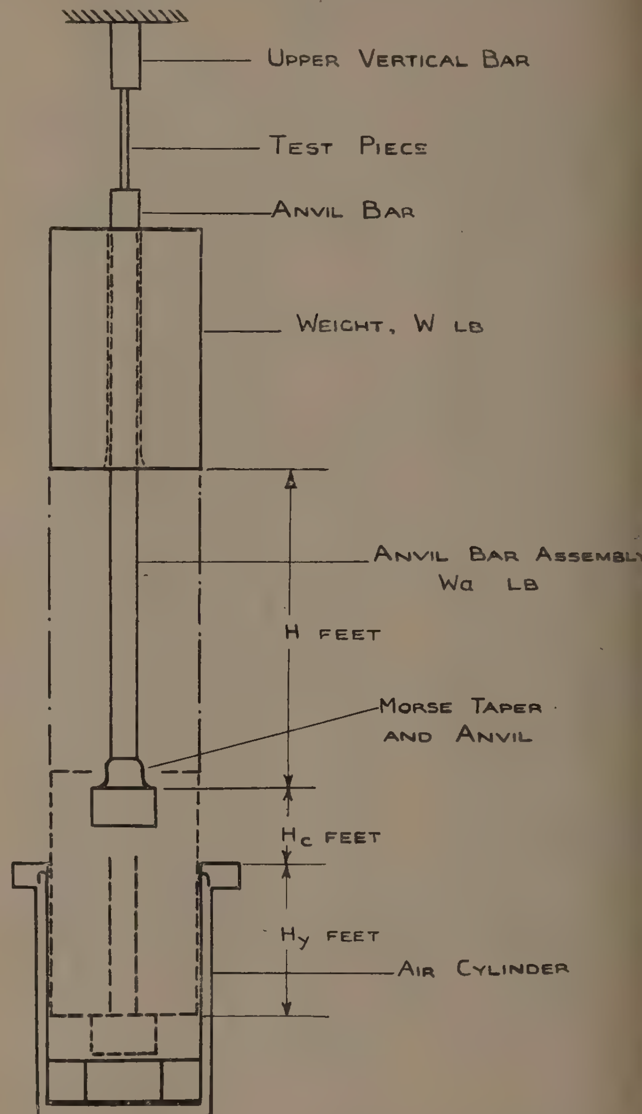

Fig. 5.—Diagram—Theory of tension impact machine

is to be tested. The grips are shown in Fig. 9, and grips and test bar are self-centering.

By means of a simple jig the 8-in. parallel gauge length is marked off at ½-in. intervals, so that wherever the neck occurs when the bar is broken in tension, percentage elongations on two different gauge lengths, each including the whole of the necking part, may be readily computed.

$$\text{Effective ductility} = \frac{p_1 L_1 - p_2 L_2}{L_1 - L_2} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

where p_1 and p_2 are the overall percentage elongations (including the necking part) on gauge lengths of L_1 and L_2. For the test bar described in Fig. 8, L_1 is normally 7 in. and L_2 is usually 1½ in., both including the whole of the necking stretch.

Autographic Tensile Testing Machine

Only a model has so far been made, but the specification of the machine which follows may be of interest and promote its detailed development.

A tensile test bar of any material or size appropriate ı the capacity of the machine is placed in the grips and ıe machine being then set in motion, the load-extension ıagram is recorded from no-load to failure or between ıy desired limits of loading.

From measurements of the test iece and from the diagram, the ield point, the effective ductility, ensile strength, reduction in area nd work done in stretching the bar re obtained.

The load is applied through the est bar to an elastic beam, the eflection of which produces a relative novement of a screw (forming the xis of a chart drum) and the chart rum (which is rotated by the relative xial movement of the screw).

A lattice multiplying mechanism applied in such a way that one art is fixed to the grips at one end f the test piece, another part is fixed o the grips at the other end of the est piece and the relative movement f these two parts is multiplied an ppropriate amount and moves a pen n an axial direction on the chart aper affixed to the surface of the rum.

The load is then recorded circumferentially and the extension axially, nd upon removing the chart from he drum a load-extension diagram is obtained with ectangular co-ordinates.

In a machine similar to the model, the frame of the nachine is vertical and supports a horizontal steel beam

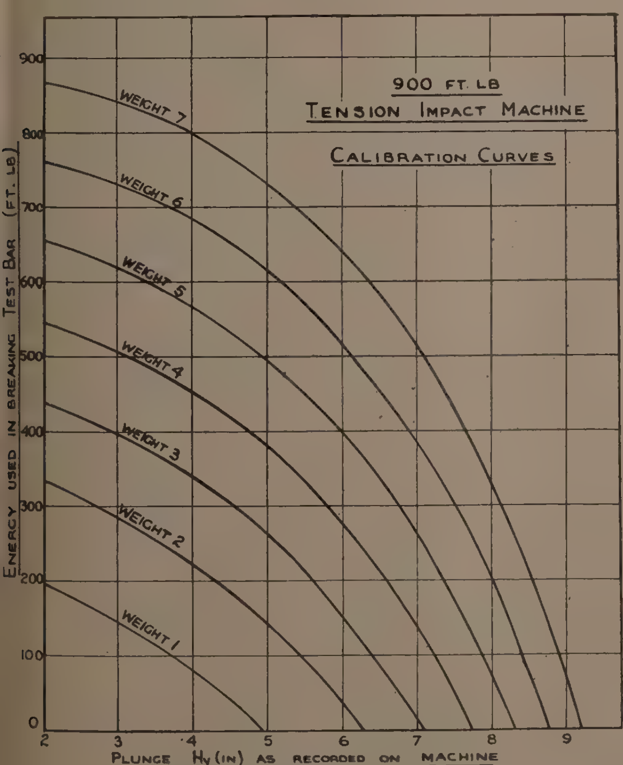

Fig. 7.—Calibration curves

at the ends with a cross piece at the centre, below which are grips for one end of the test bar and above which is a pivot upon which the chart drum spindle rotates. An Archimedian or multi-throw screw is held centrally by the upper part of the frame and engages a nut screwed centrally to the upper end of the chart drum. As the beam deflects, the drum follows it and is thereby rotated. The lower grips of the test bar are connected to a screw

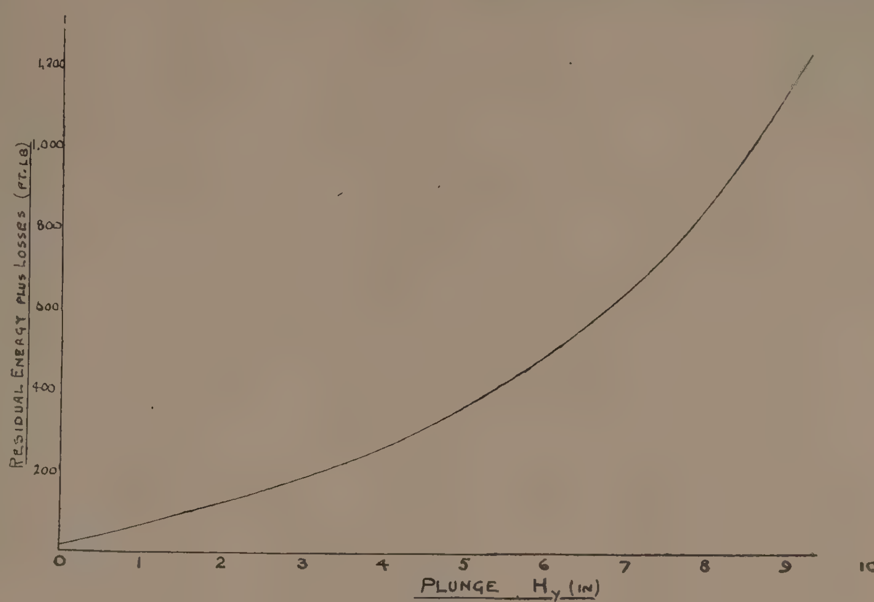

Fig. 6.—Residual energy plus losses

axial with the chart drum and in line with the centre of the beam, the screw being pulled downwards by the rotation of a nut driven by electric motor through gearing or belt, with means of setting the rate of movement of the screw to any prescribed amount. One method of multiplying the extension of the test piece is to use four bars of equal length hinged at each corner and of diamond shape, the lowest point of the diamond being fixed to the lower grips and the highest point to the pen arm, the range of movement of the pen arm being made equal to the length of the chart drum.

At the lower end of the diamond lattice a second smaller diamond lattice is formed with two additional short bars, one end of each being pivoted to a long bar and the other end of each being pivoted together, the pivot points and lengths of short bars being such that the small diamond and the large diamond are exactly similar in shape and the relative lengths of the bars are such as to give the desired multiplication. The upper point of the small diamond is secured to a slide and the slide is secured to the cross piece of the loading beam, and the slide extends upwards so as to constrain the pen arm to move axially with the drum and in line with the lowest point of the diamond lattice which is fixed to the lower grips. Then, on setting the loading mechanism in motion, the loading beam is deflected, the drum moves down with it and rotates, and when the test piece begins to stretch the lowest point of the diamond moves more than the cross piece of the loading beam and the pen arm moves upwards in exact proportion to the stretch.

The machine will be set to run at a desired speed of loading and will then record the load-extension diagram from no-load to failure without further attention.

For the ordinary "static" rate of testing, the test will be complete in under one minute, or the machine may be set for a slow speed test to take a couple of days

or a week or more, as may be desired, and be stopped and re-started at any load.

Maximum Strength in Impact Test

This is determined under impact conditions from the energy to break the test bar and the total stretch of the bar. The area under a normal stress-strain line is to a determinable scale, the product of maximum strength and total stretch of test bar multiplied by a calculable ratio, y (in the order from 0.90 to 0.97, depending on the effective ductility).

Effective Ductility (per cent.)	5	10	15	20	25	30
Ratio, y	·979	·961	·947	·938	·931	·925

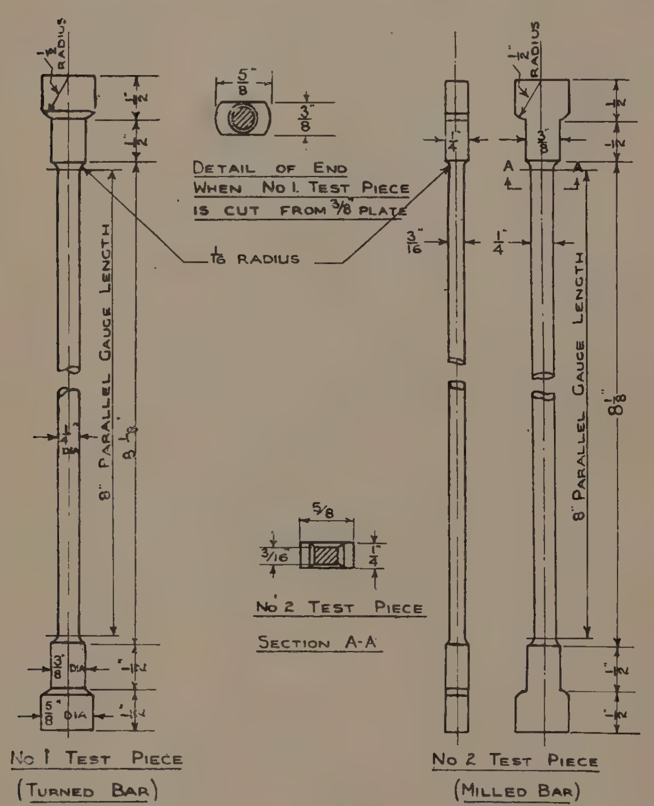

Fig. 8.—Tensile Test Pieces

The assumption is made that the shape of the nominal stress-strain line is of the same general form under high-speed testing as under static testing.

The shape of a nominal stress-strain line under static conditions is illustrated in Fig. 10.

The tension impact energy of the test bar

$$= A t_{ult}\, x\, y \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (5)$$

Where A = cross-section of test bar,

t_{ult} = maximum strength (ultimate tensile strength),

x = total elongation of test bar including necking stretch,

y = the ratio of area under nominal stress-strain line to the rectangle enclosing the diagram.

Description of Tests

Messrs. David Kirkaldy & Son, 99, Southwark Street, London, S.E.1, were commissioned by the author's company to carry out a comprehensive series of tests, and

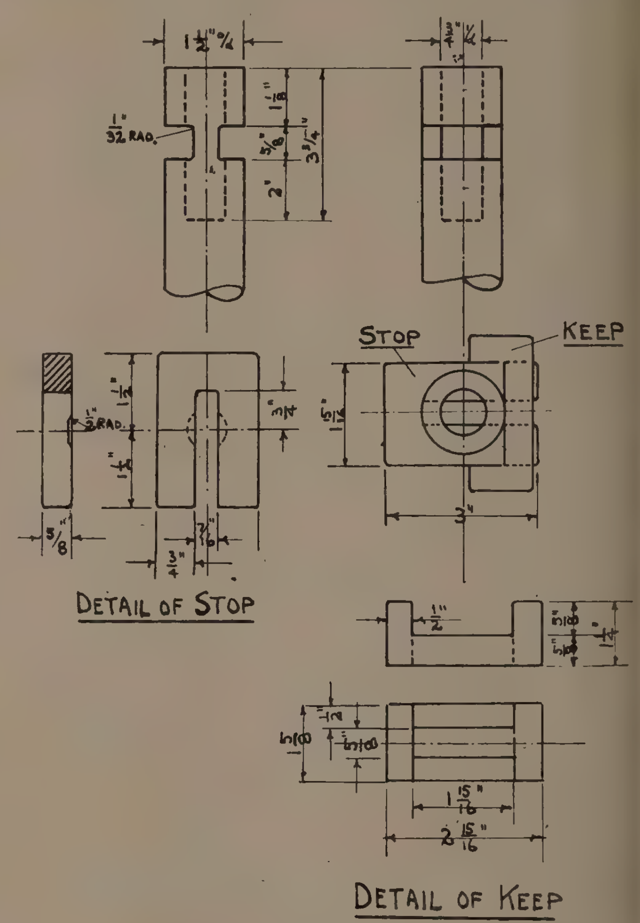

Fig. 9.—Grips

TABLE 3.—Static Tests on Steels A and B

	Steel A			Steel B		
	AX3	AZ7	Mean	BZ6	BY6	Mean
Yield point (tons/sq. in.)	35·1	36·6	35·9	29·2	30·2	29·7
Ultimate tensile strength (tons/sq. in.)	38·7	38·9	38·8	44·1	44·3	44·2
Effective ductility (per cent.) ...	8·0	12·9	10·5	12·7	13·8	13·2
Necking constant (per cent.) ...	104	79	92	92	81	87
Reduction of area (per cent.) ...	60	61	60	66	64	65
Brinell hardness No.	—	—	187	—	—	207

acknowledgement is made to this firm of testing engineers for the care and attention given to the research

Static Tests

Static tensile tests were carried out on Steels A, B and C in the direction of rolling and across the directio

rolling. A summary of the results is given in Tables 3 nd 4.

Note on Table 3

It will be noted that the amount which Steel A can be retched at "static" rate up to its maximum load arrying capacity is only half that of Steel C lengthwise, nd that Steel B has considerably better effective uctility than Steel A, though not so good as Steel C. here is a wide variation in the effective ductility of the vo specimens of Steel A.

TABLE 4.—Static Tests on Steel C

	Steel C In direction of rolling				Steel C Crosswise			
	C1.D	C1.B	C1.C	Mean	C2.C	C2.E	C2.F	Mean
ield point (tons/sq. in.)	19.5	19.3	20.1	19.6	21.0	21.1	20.8	21.0
ltimate tensile strength tons/sq. in.)	32.3	32.7	32.9	32.6	32.5	33.3	32.7	32.8
ffective ductility (per cent.)	20.7	20.4	20.3	20.5	14.7	15.5	13.5	14.5
ecking constant (per cent.	124	101	101	109	72	59	59	63
eduction in rea (per cent.)	67	65	65	66	53	48	49	50
rinell hardness No.	131	131	134	132	134	134	134	134

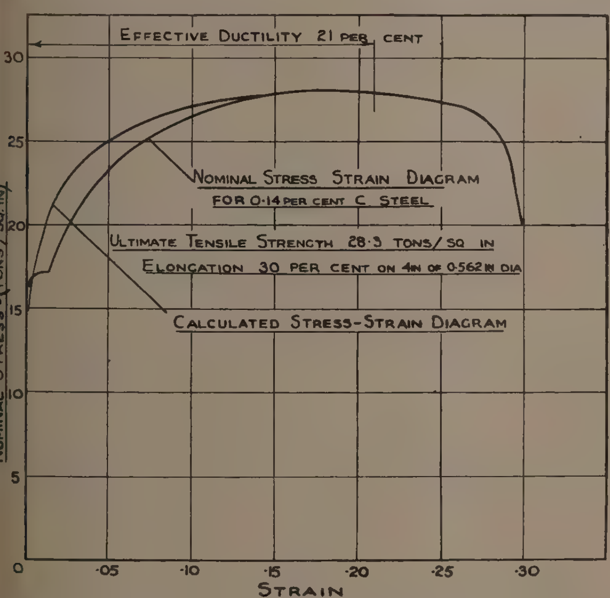

Fig. 10.—Nominal stress—strain diagram for mild steel

The necking constant is the constant k in the equation

$$p = S + \frac{kd}{L}$$

where p = percentage elongation of bar
S = effective ductility per cent.
k = necking constant per cent.
d = dia. of bar (in.)
L = gauge length (in.)

Note on Table 4

The steel plate from which all the foregoing test bars were cut had been annealed at 850°C. at maker's works. The effective ductility crosswise is approximately 15 per cent. compared with a lengthwise effective ductility of 20 per cent. approximately. It is worthy of note that this same steel was further heat-treated in

TABLE 5.—Tension Impact Results

Steel A	A.X1	A.X5	A.Y3	A.Y1	A.Z3	A.Z5	Mean
Effective ductility (per cent.)	11.8	16.5	17.2	9.1	10.4	10.0	12.5
Necking constant (per cent.)	78	82	90	84	83	99	86
Total stretch, in.	1.15	1.55	1.61	0.95	1.05	1.05	1.22
Plunge H_y, in.	7.52	6.90	6.80	8.04	8.02	8.14	—
Energy absorbed, E_b (ft. lb.)	420	530	540	330	330	300	—
Ultimate tensile strength (tons/sq. in.)	42.0	40.7	39.1	39.6	37.0	34.2	38.8
Reduction of area (per cent.)	59	60	61	53	60	61	59
Uniform energy per in. of ¼-in. dia. (ft.-lb.)	43	57	57	32	33	29	42
Necking energy ¼-in. dia. (ft. lb.)	73	72	77	73	66	69	72

Steel B	B.X4	B.Y2	B.Y4	B.Z4	B.Z8	B.W2	Mean
Effective ductility (per cent.)	13.6	17.2	14.4	13.4	15.8	18.0	15.4
Necking constant (per cent.)	89	62	92	85	34	17	63
Total stretch (in.)	1.32	1.54	1.40	1.30	1.38	1.50	1.41
Plunge H_y (in.)	7.56	6.50	7.06	7.56	7.28	7.04	—
Energy absorbed, E_b (ft. lb.)	420	590	510	420	470	510	—
Ultimate tensile strength(tons/sq. in.)	38.0	44.3	42.0	37.6	39.6	40.3	40.3
Reduction of area (per cent.)	62	63	63	65	63	62	63
Uniform energy per in. of ¼-in. dia. (ft. lb.)	43	66	53	43	54	61	53
Necking energy ¼-in. dia. (ft. lb.)	70	58	80	68	31	14	53

Steel C_1	C_1.E	C_1.F	C_1.G	C_1.H	C_1.M	C_1.N	Mean
Effective ductility (per cent.)	25.3	24.7	22.5	23.5	21.8	22.4	23.4
Necking constant (per cent.)	68	92	89	95	97	102	91
Total stretch (in.)	2.20	2.22	2.04	2.16	2.01	2.06	2.12
Plunge H_y (in.)	6.20	6.02	6.64	6.26	6.94	6.84	—
Energy absorbed, E_b (ft. lb.)	620	640	570	610	520	540	—
Ultimate tensile strength (tons/sq.in.)	33.0	33.7	32.4	33.3	30.2	30.6	32.2
Reduction of area (per cent.)	63	65	65	65	65	65	65
Uniform energy per inch of ¼-in. dia. (ft. lb.)	71	71	63	67	57	59	65
Necking energy ¼-in. dia. (ft. lb.)	45	66	62	67	62	68	62

Table 5—continued

Steel C_2	C_2.G	C_2.I	C_2.K	C_2.M	C_2.N	C_2.O	Mean
Effective ductility (per cent.)	17.1	17.3	17.1	18.4	16.9	16.4	17.2
Necking constant (per cent.)	73	72	65	74	59	49	65
Total stretch (in.)	1.55	1.57	1.54	1.67	1.51	1.45	1.55
Plunge H_y (in.)	7.14	7.44	7.36	7.50	7.70	7.78	—
Energy absorbed, E_b (ft. lb.)	490	440	450	430	390	380	—
Ultimate tensile strength (tons/sq. in.)	36.5	32.4	33.5	30.1	29.9	30.5	32.2
Reduction of area (per cent.)	51	51	47	47	45	51	49
Uniform energy per inch of $\frac{1}{4}$-in. dia. (ft. lb.)	54	49	50	48	44	43	48
Necking energy $\frac{1}{4}$-in. dia. (ft. lb.)	57	50	47	46	39	34	46

maker's works at 650°C. before being made into pressure vessels, and the effect of that heat-treatment was to give the steel cross-wise an effective ductility of approximately 20 per cent., the lengthwise effective ductility remaining at approximately 20 per cent.

A study of the results summarised in Table 4 shows Steel C (particularly when it is remembered that these results were obtained on long thin bars of $\frac{1}{4}$-in. diameter × 8-in. gauge length) to be remarkably consistent.

Tension Impact Tests

Six tension impact tests were made on Steels A, B and C_1 and seven (one was laminated) on Steel C_2. A summary of the results is given in Table 5.

The differences in plastic deformation and of tensile physical properties of the four steels under review when bars are pulled apart (1) under static load and (2) under "impact" conditions are compared in Table 6 :—

TABLE 6.—Comparison of Static and Tension Impact Properties of Steels A, B, C_1 and C_2

	Steel A		Steel B		Steel C_1		Steel C_2	
	S	I	S	I	S	I	S	I
Ultimate tensile strength (tons/sq. in.)	38.8	38.8	44.2	40.3	32.6	32.2	32.8	32.2
Effective ductility (per cent.)	10.5	12.5	13.2	15.4	20.5	23.4	14.5	17.2
Necking constant (per cent.)	92	86	87	63	109	91	63	65
Reduction of area (per cent.)	60	59	65	63	66	65	50	49
Energy absorbed per in. by $\frac{1}{4}$-in. dia. bar in uniform stretch (ft.lb.)	36	42	51	53	57	65	38	48
Necking energy $\frac{1}{4}$-in. dia. (ft. lb.)	78	72	83	53	76	62	48	46

S = Static, I = Impact

It will be noted that for the four steels under consideration, the effective ductility is more in every case at the high speed of impact loading than at static loading, the ultimate tensile strength is about the same in the case of Steels A, C_1 and C_2, and 10 per cent. down in the case of Steel B. The percentage reduction of area is hardly changed.

As might be expected from the foregoing measurements, the uniform energy calculated from static test values is a little less than under impact rate of loading, and the necking energy is not greatly different. Impact loading is not then fundamentally different from static loading in the plastic deformation it occasions, but the physical values differ numerically in most cases. In other words, plastic deformation is a function of time as well as of loading.

Points for Discussion

Both machines described are experimental, but the question is put whether it would not be worth while to examine existing impact and tensile testing methods and consider if the development of a tension impact and of an autographic tensile testing machine should not be considered from the user's point of view.

The following points are raised :—

(a) Is the Izod test all that the user wants in the way of impact test, or would he be better off with tension impact tests on plain and on notched bars ?

(b) Is the standard tensile test sufficiently comprehensive or should the steel user demand a test which gives him in addition the stretch to maximum load ?

(c) On practical consideration there is a limit to the extent to which steel may be stressed, the practical consideration being limitation of permissible deflection; but though for that reason the main members in a structure may only be stressed to 9 or 10 tons/sq. in. however high the yield point and ultimate tensile strength of the steel may be, there is every reason for using a steel with a high effective ductility in a structure to distribute the loads over the whole cross-section of members. The author would recommend as a matter of course specifying a minimum effective ductility of 15 per cent., both at static and at impact rates of loading, as a more effective insurance than high yield point against premature failure at a stress raiser.

(d) Physical measurements of steel should be made at the temperature at which the steel is to be used. The tension impact machine described lends itself to such measurements because the test bar can be placed in position with a bimetal thermometer attached, surrounded with heating or cooling pads and tested at the precise moment when the test bar reaches the desired temperature.

(e) The size of test bar used for the impact tensile research described is $\frac{1}{4}$-in. diameter × 8 in. long. For normal test purposes the bar can be shortened to 4 in. and still give sufficient accuracy for most purposes ; or if a larger machine is made, a bigger diameter bar may be used, but preferably with a ratio of gauge length to diameter of not less than 16. If small ratios of gauge length to diameter are used, the physical properties up to maximum tension load become merged in the physical properties of the necking part of the total extension.

(f) The tension impact value of a steel is much more easily comprehensible to the user of steel than the Izod value and this test can be carried out on notched tensile specimens to study the toughness of the steel at notches. The question is raised whether in fact the tension impact test is not more nearly related to practical structural requirements than the Izod test.

The Construction of the Temple Barrage on the River Lot*

By L. P. Brice

The first part of this paper describes the utilisation f "low-head" installations in the development of the ydro-electric resources of France, with a description f the principles and construction of the Stream-line 'ower Station Barrage recently completed at Temple-ur-Lot (Electricite de France).

The harnessing of low heads is shown to be a subject f some importance at the present time, owing to their igh productivity and the relatively short time taken o prepare them for operation.

In discussing the layout of low-head barrages, a omparison is made with the corresponding features of igh-head barrage layout.

The second part of the paper deals with an engineering roblem of special interest in connection with high-head arrages, namely, the driving of tunnels of considerable ength starting from a single point of attack. A practical olution to the problem of ventilating very long galleries f small cross section is described. An account is given f the progress made in the use of mechanical means for riving tunnels. A description of the work on the Nentilla tunnel in the Aude (Electricite de France) is ncluded.

Introduction

The programme which has so far been carried out since the war, in harnessing hydro-electric resources, embraces not merely the modernisation of plant which had already been in use for some considerable time, but also the construction of new works to utilise the hydro-electric resources offered by both mountainous and low-lying regions.

At the present time in France a series of low-head power station barrages is in course of completion ; the high kilo-watt-hour productivity offered by these power stations at a comparatively low cost justifies the interest felt in their adoption.

We shall first of all set out the general principles and describe the construction of a plant of this type ; the stream-line power station barrage recently completed at Temple-sur-Lot (Electricite de France).

Furthermore, the operations now being carried out to harness high heads include a series of works where the construction of underground galleries on the tunnel-system has been proceeding on a considerable scale ; nowadays, when we wish to join the basins supplying water from two valleys, we no longer have any hesitation about driving a tunnel through the group of mountains which separate them. The contracting firms concerned therefore have to study the problems arising in the construction of tunnels with a very considerable length, perhaps well above 10 or 15 kilometres. These in turn lead to new problems connected with the ventilation and the time taken for the mechanical work of driving the tunnels.

Secondly, we shall describe the recent solutions adopted, with particular reference to the Nentilla Tunnel, in the Aude (Electricite de France).

**Paper read before a Joint Meeting of the Institution of Structural Engineers and the British Section of the Societe des Ingenieurs Civils de France at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 13th December, 1951.*

I—Low Head Barrages

The Electricite de France are now going ahead with their programme for utilising the hydraulic resources of the country. Where we have a wide range of varied possibilities, where hydraulic operations are concerned, the installations chosen may be divided among the high and medium heads or allocated to the low heads, and our choice will first and foremost be guided by the following principle : the electric power produced by a hydro-electric power station is proportional on the one hand to the head utilised and on the other hand to the total flow which passes through it.

The high head works where achievements have been so spectacular as to attract public attention, catch very small flows, such as 14, 12, 10 or 4 cubic metres per second, utilised at heads of considerable height such as 500 to 1,500 metres, the medium heads have flows of the order of 200 cubic metres per second for heads varying from 50 to 200 metres ; low-head works, with a height of not more than 50 metres absorb a very considerable flow, ranging from 200 cubic metres to more than 1,500 cubic metres per second. (In power stations set up recently, heights of 20 metres are quite usual ; power stations now open only use heights of 10 metres, or even 8 or 7 metres.)

If we consider an installed power of the same order (30,000 kilovolt-amperes) giving a more or less equivalent output (about 90 million kilowatt-hours) we see that a high head dam (such as GEDRE, Gave d'Heas, in the Upper Pyrenees Department) utilises a flow of 4 cubic metres per second with a height of fall of 672 metres, while a streamline dam (such as that at Temple-sur-Lot, in the Lot) only has a height of fall of 10.8 metres for a flow of 293 cubic metres per second.

In practice, however, the high-head will only give its peak output spasmodically, while the low-head will do so continuously, except during the brief low-water periods.

Let us consider some more powerful plants ; for an installed power of 160,000 kilovolt-amperes, a high-head power station—that at Pragneres, in the Upper Pyrenees —will give 243 million kilowatt hours for a head of 1,254 metres and a flow of 14 cubic metres per second ; the same installed power with a low-head (at Ottmarsheim) where the head is 16.4 metres but the flow 1,080 cubic metres per second, will furnish 900 million kilowatt hours.

Still higher powers are now being installed ; the 225,000 kilovolt-amperes with the medium head at Bort-les-Orgues, in the Dordogne, where the head is 113 metres, with a flow of 200 cubic metres per second, will produce 327 million kilowatt hours ; at the low-head power station at DONZERE-MONDRAGON, in the Vaucluse, for a head of 23 metres, utilising a flow of 1,530 cubic metres per second, the 300,000 kilovolt-amperes to be installed will produce 1,980 million kilowatt hours.

These extreme figures, quoted objectively, may throw some light on the nature of the two types of installation.

Utilisation of Low-Head Rivers

In a country like France the utilisation of low-head works now tends to occupy the main place in hydro-

electric production, the complementary factor being the high-head reservoirs. It is only thanks to the contributions of the thermic cycle, however, that the over-all output required by national needs is on the whole attained.

In a country where, owing to the natural wealth of mining resources, thermal power stations are the principal means of generating electric power, and where, moreover, with a high-head of water, the possibilities as regards the flow would be limited, it would appear that the idea of setting up a system of low-head power stations would merit favourable consideration. During the brief seasonal low-water periods, hydrological conditions in which rain predominated would reduce the contribution required from the thermal power stations.

The continually increasing demand from consumers necessitates new installations, which have to be set up as a matter of urgency. It would seem that the demand could be satisfied by adopting those lay-outs which require the shortest period of preparation and which produce a great number of kilowatt-hours at the lowest cost.

High-head plant involves special constructional requirements :

Owing to the Incidence of the Altitude :

- Access-routes have to be created ;
- The personnel have to be lodged and fed ;
- Sufficient space has to be provided, in the vicinity of the main working site, for the auxiliary plant ;
- The heavy material has to be transported and placed where it is required.

Further, owing to the Multiplicity of the Works to be Constructed :

- The barrage proper ;
- The underground gallery ;
- The penstock ;
- The flood-weirs and restitution channels ;
- The pumping stations, water-chambers, balance-vents, etc. ;
- The power station itself ;
- Dwellings, etc.

This work is carried out either concurrently or item by item, according to the topography of the site selected. The most recent technique, known as the " ski-jump system " is to combine the barrage, the power station and the weir in one single block. This simplification of the work may offer practical advantages later on, although it raises some complex problems for the builder, where design and construction are concerned.

Low-head plant, with its low-lying situation, is easy of access. It does not involve the onerous work of tunnel-driving or the construction of pipe-lines ; labour is recruited without difficulty ; due allowance being made, the work in such power stations is carried out at a more rapid rate of progress.

The building site has the advantage of forming one continuous block. The power station buildings and the piers are next to one another.

On the other hand, to create the head we have to raise the old water-level upstream ; which involves additional work to ensure the protection of the banks.

A new idea has been adopted in low-head technique : the " pier power station." This is what we find at the Castets dam, in the Lower Pyrenees Department, now in course of construction. There are two independent water-chambers inside one single pier : in each chamber an armoured, oil-pressure, inclined-axle, single block Kaplan turbo-alternator unit will operate under constant immersion. The control and maintenance of the machinery is carried out by means of an elevated gantry placed on the top of the pier.

This system means a saving in the constructional work involved for the usual power station and its machine room.

This prototype which in the present case only deals with a low power, may open the way towards still more powerful and economical plants in time to come.

The conditons under which high-head and low-head installations are constructed are too dissimilar to allow of the choice of one type to the exclusion of the other. Assuming uniform hydrological conditions justifying the adoption of one single type of head, the cost price per kilowatt-hour generated seems to us to be the determining factor.

Reliable documentary data available suggests that this cost price is as follows :

High-Head : 67 to 180 French francs (in June, 1951)
72 French francs (average for the most recent installations).

Low-Head : 20 to 42 French francs
30 French francs (maximum average for the most recent installations).

We will supplement these basic data by describing the principles and construction of a low-head barrage power station completed recently.

Stream-Line Power Station Barrage at Temple-sur-Lot

Geographical Position

Downstream from Villeneuve-sur-Lot, the Lot is a calm river 150 metres wide, which winds between the hills of the Castelmoron region as far as Aiguillon, where it flows into the Garonne. Very old barrages on stone bedding form the limits of reaches of 2 to 3 kilometres in length and provide the mills and old power stations with a fall of a few metres in height.

Hydrological Data

The catchment area covers 11.194 square kilometres.

The rate of 16.5 litres per second corresponds to an average flow of 183 cubic metres, and it varies from 20 to 40 cubic metres per second in the low-water season in summer, reaching 4,000 cubic metres per second in the winter flood period. The plant may receive up to 293 cubic metres per second.

The advantage of this barrage may be easily imagined since it is in the winter-time that it is utilised to the full and since this is the period when the demand for power is greatest.

For a utilisation-rate of 3,700 hours a year the output amounts to 90 million kilowatt-hours.

Topographical and Geological Conditions

The fall applies to a 40-kilometre section of the course of the Lot. The height, 10.8 metres, has been selected so as not to flood the riverside agriculture upstream.

The average gradient of the bed of the river is 0.2 per kilometre.

Its geological composition includes a bed of alluvial c 3 to 4 metres resting on a bank of very hard mar (9 metres thick).

In particular, the phenomenon of an undergroun water-sheet starting upstream but without outflow wa met with, and this unusual obstacle was surmounte when the foundations of the power station were con structed.

Description of the Works

The Temple-Castelmoron power station is of the combined barrage-and-power-station type. The barrage 100 metres wide) occupies the right bank, while the power station (60 metres wide) is situated on the left bank. (See Fig. 1).

The level required by regulations is 39 metres, and the "restoration-level" is fixed at 28.2 metres, giving a maximum fall of 10.8 metres.

Barrage

The barrage is of the straight-sluice type with an opening of 20 metres between piers, and with a 10-metres "retention."

The sluices are four in number, and rest on piers with a maximum width of 5 metres and a length of 18 metres. The end-piers, on the left bank, serving as a support to the gable wall of the power station are the more important.

The sills of the sluices are at the level of 29.00 metres, while the foundations of the piers and the floors rest on the marl (level 23 to 24).

The piers are surmounted by a superstructure accommodating the control-cabin and the counterweights of the sluices.

Power Station

The power station includes a building accommodating three vertical-axle Kaplan turbo-alternator units : two of 14,500 kilovolt-amperes, supplying three-phase current at 5,500 volts ; stepped up to 60,000 volts, this current feeds the main interconnection-system ; one auxiliary unit of 1.600 kilovolt-amperes feeds the internal services of the power station and the local systems at 13,500 volts.

There is an annexe containing the erection shop, the transformer-dismantling station, the workshops and general services.

The total power is 30,600 kilovolt-amperes.

Civil Engineering Works

The whole unit consists of six separate foundation-blocks. In the part of the building situated downstream from these units there is a platform supporting the transformers, circuit-breakers and disconnecting-switches of the 5/60 kilovolt step-up transformer station and the transformers for the local supply system.

On the upstream side there is a platform 62 metres long for the grid cleaner to be moved about where required.

Fig. 1 Aerial view of Power Station and Barrage

Each turbine is fed by three openings of 4.4 metres (free space) and 13.5 metres in height ; each opening has two exactly similar cofferdams.

The auxiliary turbine is closed by one single cofferdam.

Under the terrace there are seven hydraulic-wall type containers for the storage of transformer oil.

Execution of Work

The work was started in 1947 ; the barrage put in operation in November, 1950.

Seven prefabricated buildings were set up on the left bank, to the right of the access routes, to house the staff. Eight separate hutments for the staff were constructed on the right bank. (See Fig. 2).

All the installations for the working site were grouped together on the left bank in the immediate vicinity of the future power station.

A quarry was opened some distance away on the other bank.

The installations included :

General services (storehouse; workshops, offices, yards);

Two suspension cableways (span 230 metres ; hook-power 5 tons) ;

Concrete works (main concrete works Richier S.13—concrete-mixer of 1,300 litres) producing 300 cubic metres per day.

Kaiser crane (power : 1.5 tons at 20 metres) for making the superstructures.

The total amount of concrete dealt with was 35,000 cubic metres, 17,000 being for the power station.

Fig. 2 General view of site at commencement of operations

Equipment for pile-driving and extraction, mounted on pontoons.

An overall length of 1,150 metres of piles were driven (Larsen III or Frodingham type), each weighing 400 kilog. in lengths of 10 metres.

The excavations and earthworks carried out have been estimated at 75,000 cubic metres.

A primary cofferdam enabled the power station and the attached pier to be constructed. The presence of the underground water-sheet mentioned under "geological conditions" necessitated the construction of a double belt of well-points to lower the water level.

In 1948, while the work was being carried out on the left bank, the cofferdam on the right bank was proceeded with, and thus permitted the construction of piers 3 and 4, the intermediate floor and a supporting-wall for the upstream and downstream banks. (See Fig. 3).

This cofferdam was extracted in the winter of 1948-1949.

The next stage of the work resulted in the completion of the following three operations :

A primary cofferdam closing on pier 3 (already constructed) enabled floor 3 and pier 2 to be carried out.

Utilising the same part, parallel to the Lot and closing on the one hand on the power station pier downstream and on the other hand on the upstream screen of the power station cofferdam, a second cofferdam enabled pier 1 and floor 1 to be constructed. Earlier the power station cofferdam had been extracted with the exception of the upstream screen ; thus the downstream part of the power station was in water, its upstream part stil remaining dry, thanks to provisional concrete barrier which closed the outlets from the diffusers.

Operations having proceeded satisfactorily, it wa found possible to complete the barrage by constructin the floor before the end of 1949. For this purpose, th central part of the screen parallel to the river wa extracted and a closure made on the two adjacent pier

In 1950, the superstructures of the piers and the foot bridge connecting them were completed.

Equipment :

On July 10th, 1949, the first third of the power statio building being completed, was handed over to the stee

work contractors, who immediately erected the overhead crane. It was thus possible for the various parts of the turbines and alternators to be placed in position without delay.

The modification of the upstream and downstream water-level had earlier necessitated considerable additional work:

Demolition of an old barrage on stone bedding.
The Gresille and Fougrave pumping-stations.
The construction of various protective devices.

The date specified by the Electricite de France for the completion of the work was complied with, thanks to intense efforts on the part of the personnel and the adoption of prefabrication methods.

We have to reduce this risk to a minimum by installing an efficient system of ventilation which will enable the personnel to work, in complete safety, and so obtain the maximum output.

Dust

The dust is produced :
During drilling.
During the blasting.
During the removal of blasted material.
In the galleries, by reason of draughts.

The degree of dust-proportion can be evaluated from the weight of the dust collected in a filter. This is an empirical process which fails to give results of any use, since the weight of the large particles of dust bears no fixed proportion to the weight of the fine particles (for a large particle of 10 microns weighs one thousand times as much as a small particle of 1 micron).

Fig. 3 A Cofferdam in construction

II—Driving of Tunnels
Improvement in Ventilation and in Hole-Blasting Methods

The construction of high-head hydro-electric plant involves a variety of practical problems where the driving of tunnels is concerned :

Certain of these problems are concerned with the creation of an atmosphere physiologically favourable to the output of the personnel.

Others concern the selection of the mechanical means for the work.

I—Surroundings

Working underground is in general physiologically undesirable. The air is polluted on the one hand by the dust caused by the drilling and on the other hand by the gas given off by the explosions, so that the atmosphere may be injurious.

It is only the fine particles that are harmful ; they have to be evaluated with precision by the numerical process.

There are a number of dust-counting apparatuses in existence ; conimeters, impingers, thermal precipitators, soluble filters, most of these being manufactured abroad and each presenting its own peculiar advantages.

In France the Sub-Committee on Mines has adopted a dust-counter of the soluble filter type, using tetrachloronapthalene, enabling exact granulometric data to be obtained. M. Sauzeat, the Director of the Dust Research Laboratory of the St. Etienne Collieries, recognised as authoritative, uses a thermal counter.

The numeration of the fine particles (5 microns to two-tenths of a micron) is an indispensable process, as they penetrate as far as the pulmonary alveolæ, where

they may be absorbed. When they float in the air they form the dangerous element of dust-clouds, constituting what are known as "aerosols." Their behaviour is exactly the same as that of gases.

Numbering methods have revealed that air, after dry-boring, has been carried out, contains 5,273,000 of them in each litre.

GAS

The second pollution-factor is constituted by the gases released by hole-blasting. They form a cloud 40 to 60 metres in length, which flows back and forth along the gallery. In addition to large dust-particles, this cloud contains a mixture of fine particles and noxious gases forming a complex "aerosol" known in practice as a "plug."

A kilogramme of dynamite releases .8 of a cubic metre of gas at a temperature of 0° centigrade and at a pressure of 760 mm. of mercury. In the case of BAM dynamite the proportion of carbon monoxide is 5 per cent. (7 per cent. in the case of cartridge surrounded by paraffin paper).

A proportion as low as 0.0005 per cent. is noxious. Regulations lay down a proportion of 0.0002 per cent. as the maximum tolerable underground.

EVACUATION

The dust is evacuated by ventilation. (See Fig. 4).

It will be noted that some of the dust is laid by the water (moist gallery, screen of water, watering the excavation-waste, injection of water in the boring-hammers) but the fine dust for which a long humidification-time is required (the time being in inverse proportion to the square of the diameter of the dust) is not successfully laid ; ventilation is therefore required in all types of tunnel ; the ventilation has to be designed with particular care in the case of very long tunnels without intermediate openings and with a small cross section (about 5 to 7 square metres).

Ventilation in the Nentilla Tunnel in the Aude

This tunnel has a length of 6 kilometres and a cross section of 7 square metres. It could only be tackled at the ends. We shall consider the 3-kilometre section, for which the firm of Sainrapt & Brice was responsible.

The ventilation-plant installed at the entrance to the gallery comprises two ventilators enabled by a by-pass to function either by suction or by pressure and one ventilator. (See Fig. 5).

The two ventilators with which the Nentilla station is equipped are of the "Ventil" type, H.P.250.850, 80 h.p., rotating at 2,958 revolutions per minute, with an output of 2.3 cubic metres per second at a water-pressure of 1,320 mm.

The aero-ventilator is a "Rateau A.40," operating at an effective pressure of 6 kilos to the square centimetre, rotating at 5,000 revolutions per minute and giving an output of 2.3 cubic metres per second.

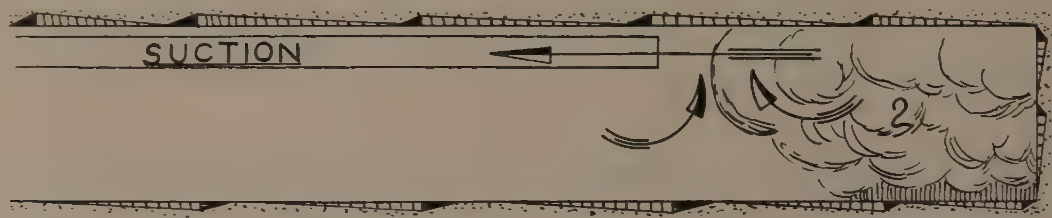

Fig. 4—Method consisting of suction alone

The ventilation-piping consists of air-pipes of 400 mm. in diameter, assembled by special hermetic, non-projecting joints and secured by flat pins. They are simple to assemble and can be used again. These patented joints have been in use since 1948.

As regards the geological nature of the tunnel, it is of hard granite requiring 26 kilos of explosive per charge.

The ventilation methods adopted in succession were : suction only, pressure only, suction followed by pressure, suction combined with an aero-ventilator.

The results obtained with the first three methods were considered inadequate for rendering the atmosphere of the gallery sufficiently healthy ; the last method was found effective from the point of view of the evacuation of all the harmful elements in the air and also on account of the shortness of the time required for their evacuation.

The alternative operation properly timed of the main suction-conduit in alternation with the aero-ventilator used as a compressor completely overcomes the carbon-monoxide-saturated "plug" and the mass of the "aerosols."

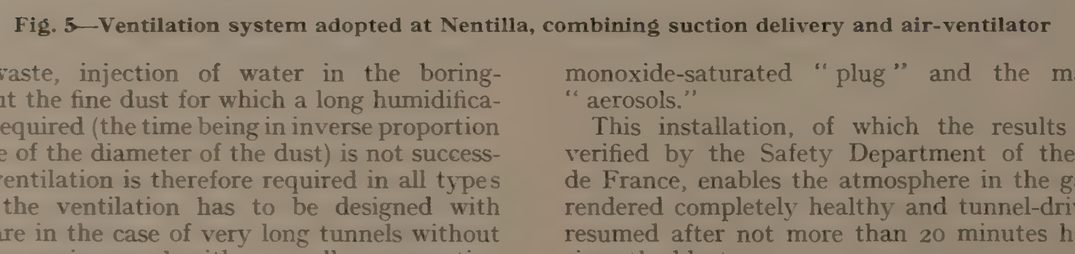

Fig. 5—Ventilation system adopted at Nentilla, combining suction delivery and air-ventilator

This installation, of which the results have been verified by the Safety Department of the Electricité de France, enables the atmosphere in the gallery to be rendered completely healthy and tunnel-driving can be resumed after not more than 20 minutes have elapsed since the blast.

Selection of Mechanical Means for Tunnel-Driving

The cycle of operations involved in tunnel-driving is as follows :

Preparation for boring.
Boring.
Charging and blasting.
Ventilation.
Removal of blasted material.

These basic operations are subject to special difficulties arising from the site, the nature of the rock, the cross-section of the gallery, the means used and the material available.

In each of these operations considerable improvements have been noted within quite a short period of years. We have examined the system of ventilation in use in the Nentilla Tunnel, this being considered as one of the most efficient, and we shall now examine the tunnel-driving operation itself, and here again we shall be concerned with the underground works at Nentilla, in the Aude.

Tunnel-Driving Material

At the end of 1947, ordinary steel bits with rosette-form cutters were used, (see Fig. 6), either on individual supports or on a "Jumbo," a mobile metal frame on rails, fitted with four boring-devices. The weight of the hammers, all of the axial water-injection type, ranged from 20 kilos for ground of average hardness to 60 kilos for hard ground intersected with faults.

The normal wear-and-tear on the edges necessitated frequent cooling and re-forging, with the use of pneumatic forges ; these repair operations, indispensable if the boring-work was to be carried out accurately, slowed down the work and had to be carried out several times before the depth selected for the blast-hole was reached ; the spare parts also had to be loaded on to a lorry attached to the "Jumbo."

Thus, lengthy manipulations and the considerable cluttering-up of the gallery reduced the rate of advance to a few metres per 24 hours, peak-performance of about 5 metres being exceptional.

In the second half of 1948 new equipment was put into use, consisting of Cormorant bits on Atlas-Polar hammers, fitted with a tungsten carbide single bevel edge.

This light apparatus, weighing 21 kilos, with its high percussion-frequency (2 to 3000) enables the boring to be performed with regular acceleration.

It may be estimated that this apparatus, which is of Swedish origin and which comes from the Sandvik Steel-works, gives an economic efficiency 25 to 30 per cent. above that obtained with the old methods, whatever the nature of the ground, if an equal amount of labour is employed.

The following is achieved in the organisation of the works :

Standardisation of the material : one single type of hammer and bit.

Elimination of the "Jumbo" and the lorry for transporting the spares.

An economy of labour (five men, as against 12).

The labour does not have to be selected so exactingly (as the apparatus can be easily handled by an operator in normal physical condition).

An economy in "dead time" (two normal shifts of four hours per gang of five men).

The new material is placed in position when the gang is relieved.

A saving of free space in the gallery, rendering the surroundings more healthy.

A regular rate of feed.

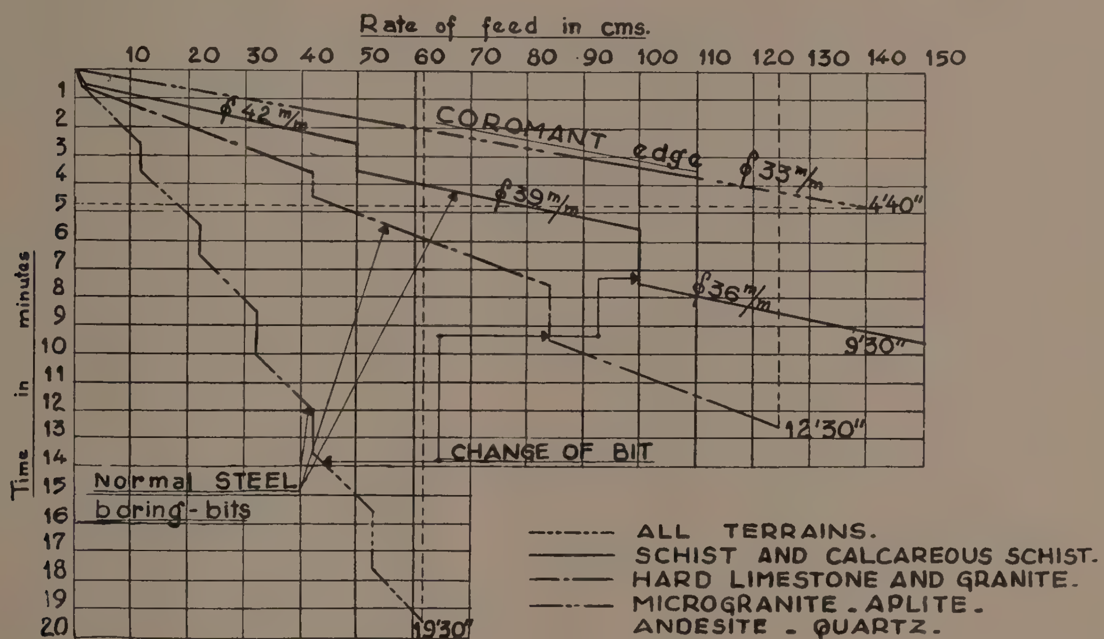

Fig. 6—Comparative diagram of rates of feed

The adoption of tungsten carbide tunnel-driving equipment, in conjunction with the ancillary improvements, such as the electrically operated firing method, the use of delayed-action electric detonators, the accelerated ventilation, the use of special buckets with shutters, and the pneumatic discharging method, have enabled a complete cycle of work to be carried out at Nentilla in not more than four hours.

In practice, three gangs of five men, each working eight hours, carry out six blasts in 24 hours. This rate of work is capable of still further improvement.

Here are a few figures obtained from recent observations :

In the gallery at the Nentilla works, in the Aude, the Tuc des Campets section, measuring 2,995 metres, without window, with a cross-section of 7 square metres —the ground consisting of limestone and shale, with influxes of water—was carried out at an average daily rate of five metres (including all operations such as timbering, sheeting and clearance of waste).

The maximum daily rate of advance was 14 metres and the maximum monthly rate of advance 171 metres.

In other Pyrenean works average daily rates of advance of 7 to 9 metres are met with in unfavourable ground, the rates attained on favourable ground being 8 to 11 metres.

In connection with these observations it may be said that the construction of the tunnel—still regarded, only recently, as a factor which slowed down the work of harnessing high falls—is now an operation in which we may normally expect to witness a considerable speeding-up, by reason of the improvements made in the material used.

DISCUSSION

The technical business of the joint meeting was the presentation and discussion of the above paper. Unfortunately, owing to fog, Monsieur Brice was unable to reach London, as he had hoped, and the paper was presented on his behalf by M. de Jarny.

At the invitation of the President of the Institution, Mr. R. C. S. Walters (President of the British Section of the Société) and M. de Jarny accompanied him on the platform.

The President extended a hearty welcome to the members of the Société and said the Institution was always pleased to offer any service it could in such matters, to enable them to carry on with their good work. Their joint meetings had always been worth while.

He had hoped, he continued, to have been able to introduce the author of the paper, but unfortunately that could not be. He understood that M. Brice had remained at the aerodrome at le Bourget from 9 a.m. until 4 p.m., with no results regarding the departure of aircraft, but eventually he had to give up. The meeting could congratulate him on having tried, and commiserate with him in his disappointment. The meeting was fortunate, however, in the fact that M. de Jarny had agreed to present the paper ; and in welcoming him, the President said it was the fourth occasion on which he had presented a paper on behalf of an author from across the Channel.

M. de Jarny presented the paper, which he illustrated by means of a number of lantern slides and two cinematograph films, showing the construction of the Temple Barrage, the plant installed in it, its appearance on completion and the nature of the countryside.

A third film illustrated a semi circular barrage constructed in the Pyrenees. Its height was 120 ft., and its construction resulted in the creation of a very large artificial lake.

The President expressed thanks to M. de Jarny for the fascinating way in which he had presented the paper, and to M. Brice for the very considerable trouble he had obviously taken to prepare the paper. It was, he said, most interesting, informative and well documented, and M. de Jarny had taken it over in a manner which was very entertaining.

Mr. R. C. S. Walters, seconding, said that M. de Jarny had presented the paper extraordinarily well ; apparently he knew the site of the Temple Barrage very well and had been concerned with the work. In addition, he had presented three excellent films to reinforce the valuable information given in the paper.

(The vote of thanks to M. de Jarny and M. Brice was accorded with acclamation.)

M. de Jarny briefly responded.

Mr. Peter Scott congratulated the contractors responsible for the construction of the Temple Barrage on having completed the work on time—an achievement which was seldom seen in this country nowadays ! He did not enter, however, into the reasons for it.

Asking for information of the contract price and the actual cost of the work, he said that we in this country were troubled with continued inflation, and his experience was that there was an increase of 7-10 per cent. in costs in each succeeding year, which doubled the disadvantage of being unable to complete the works on time.

The section of the paper dealing with the tunnels interested him, particularly in view of the speed of advance achieved in driving them. In this country great speeds, even spectacular speeds, could be and had been achieved in the driving of small tunnels, but once we came to the big tunnels the speed of output seemed to drop well below that which he had heard was achieved in America. The reason seemed to be that the small tunnel could be bonused and the workmen would get into a good swing and would produce spectacular results, but once we began to open out the tunnels to larger diameters the bonusing became more difficult and the output dropped accordingly. The tunnels described in the paper were small, presumably because of the high head available, which required so little water.

Next, Mr. Scott commented on the emphasis which was laid in France on the tunnel ventilation problem. In all the tunnels he had dealt with in this country, he said, ventilation had not been a major problem ; we always ventilated artificially as a matter of course until a tunnel was holed through, and then we had the advantage of natural ventilation. Probably a reason why ventilation was not a major problem was that wet drilling was used, for the water, of course, did lay the dust very considerably.

Taking the paper as a whole, the point of greatest interest to Mr. Scott was the author's attack on the high head scheme. M. Brice was a champion of the low head scheme, and the one discussed in the paper was most interesting. We in this country, had not the same opportunity to develop low head schemes, because our rivers were not of sufficient volume ; very few of our rivers could be treated in the way M. Brice had treated the river Lot.

Whilst Mr. Scott believed that the low head scheme described might be more economical than the high head scheme referred to, he felt that perhaps that statement required some qualification. As M. Brice had said, the amount of power produced was the product of the quantity of the water and the head of the fall. But another consideration was the load factor of the station and the low head scheme described seemed to work on 40 per cent. load factor or something of that nature. He asked what was the load factor of the Temple Barrage station.

With low head schemes of that nature, to which we in this country usually referred as "run of the river" schemes, we had either to install very little plant sufficient to deal with the low flow, in which case we lost a lot of water power during the periods of high flow and medium flow ; or we might install plant for the average run of the river, in which case part of the plant was idle during dry weather and a lot of water ran to waste during the flood season ; or we could install a great deal of plant to take advantage of the high flood, much of that plant being idle during the rest of the year.

The same sort of consideration applied, of course, to the high head schemes. Mr. Scott felt it was for reasons of brevity that the author had confined his remarks to high head schemes where it was not possible to provide reservoirs at the upper ends of the catchments ; and his remark about being unable to maintain a continuous output would be correct, of course, in such cases. In Scotland we resorted to the reservoiring of schemes

the utmost extent, so that we could use every cusec of the small amount of water available to us. Most of those schemes were such as M. Brice would term medium head schemes, of 500-800 ft. head, and in most cases they operated at a very high load factor. Schemes used for aluminium production operated at as nearly as possible 100 per cent. load factor, because they were required for continuous metallurgical processes throughout the 365 days of the year. In the case of the schemes being constructed for the Scottish Hydro-Electric Board we had the benefit of the grid ; nevertheless, most of them were to operate at fairly high load factor. But a few, such as the Sloy and Galloway schemes, had the advantage of using a high flow for a short period, the balance being made up from thermal power stations supplying through the grid.

Perhaps M. Brice was a little unfair to the high head stations, in that he had not given them the benefit of having reservoirs. However, probably that was because he did not wish to go into too great detail.

Commenting on a remark by M. de Jarny concerning the suitability of the African rivers for low head development, Mr. Scott said he did not know which of the African rivers were envisaged, but those with which he had had anything to do were far from having steady flow. He had in mind particularly a river in the Gold Coast having an average flow of something like 40,000 cusecs, its dry weather flow being only about 2,000 cusecs ; that would not be suitable for an unreservoired site. Similarly, most of the other rivers—the Zambesi, the Niger, the Kafue Congo, and so on—appeared to suffer in the same way from the high evaporation during the dry season, which made the difference between the low flow and the average flow too great, he imagined, to allow low head schemes to be economical.

Mr. W. Hawthorne expressed his gratitude to M. Brice for the trouble he had taken in preparing the paper, and congratulated him on having so admirable a deputy, who had delivered the paper in a most excellent manner. It was a great pity that M. Brice was not able to reach London, so that he could have seen how very much the meeting had enjoyed the paper.

The speaker said he had gathered that the intake to each turbine was divided into two channels, and he presumed that each channel had some sort of gate which could be closed down when it was necessary to de-water a turbine. He would be interested to know what kind of gates were used. There was a pair of gates for one turbine. In a station where there were several turbines, was there a gate in each channel, or did one pair of gates serve for all the intakes ?

Secondly, were the gates capable of being lowered against the full runaway flow, in case a machine suddenly lost its load and the turbine guide vanes could not be closed owing to some defect of the governor servo motor ?

Thirdly, how were the gates released in emergency ? Were they released electrically by remote control from the station, or were they released manually by operating the head gear ?

Coming to the driving of tunnels, Mr. Hawthorne asked what precautions are taken in France against premature explosion of the blasting charges when there are thunderstorms in the neighbourhood ? In this country, he said, fatal accidents had occurred owing to the charges being detonated during thunderstorms before the firing arrangements were completed and the workmen withdrawn.

Finally, he asked whether in France there had been trouble due to the corrosion of concrete by the water in hydro-electric schemes, and what kind of cement was used for facing the concrete linings of the tunnels.

Mr. Faulkner Nuttall, as a visitor, joined in thanking both M. Brice and M. de Jarny for the excellent stimulation they had given us to re-investigate some of the low head schemes, which usually were put on one side.

Being aware that the French were investigating the Niger river, he asked what effect schemes may have on the flow as there is a possibility of a low head scheme on the lower part of the river in Nigeria.

Mr. R. C. S. Walters, discussing provision for flood water through the sluice gates, said that he was concerned with waterworks dams and it was the practice in that sphere to provide overflows which, without involving any manual labour whatever, would take the maximum of any flood which might occur in a thousand years. He assumed that at the Temple Barrage on the River Lot there was some control system which would operate the sluice valves, and he asked what depth of water would flow over the concrete sill at the bottom of the sluice gates.

In this country, he continued, we regarded a low flow dam as one dealing with a fall of up to 12 ft., whereas in France a low flow dam would be one for a fall of about 10 metres. Examples of low dams here were those at the West Hampshire waterworks, near Christchurch, where the fall was 3 or 4 ft., and in the Chester scheme, where the fall was about 12 ft. At the West Hampshire works, hydraulic power was used in connection with the waterworks, but the Chester scheme was purely for generating electricity, although the nearby waterworks used some of the electricity.

There was no doubt, he continued, that low pressure schemes were becoming of greater importance, and that was why M. Brice's paper was so interesting to him and to other water engineers ; they were using the low heads in connection with their ordinary water supply dams for generating current to be used in and about the works.

Apparently there was no navigation on the River Lot, for there was a dam there before the new one was erected. In this country the necessary " paraphernalia " had been referred to ; we should want a lot of paraphernalia to block even some of our minor rivers, which might be useful, and we should have a great deal of difficulty in acquiring rights.

Broadly speaking, he said the Report of the Water Power Resources Committee (1921) had rather suggested that there was no dam site in this country worth developing for electric power generation. Nowadays, however, owing to the difficulty with regard to coal supplies and to increased costs, it would appear that that view might have to be revised.

Finally, as an old Appleby Frodingham boy, Mr. Walters added his thanks to M. Brice and M. de Jarny.

M. de Jarny replied to some of the points raised in the discussion, but deferred his reply to others until he had obtained the necessary information from M. Brice.

First, in a reference to Mr. Scott's comment that costs of construction in this country were increasing at the rate of 7-10 per cent. each year, he said that in France they would be delighted if the increase of costs each year were only 10 per cent. The rate of increase there was in fact far more than that, and in addition it was very irregular. Sometimes, as in 1949-50, costs would remain fairly stable for a period approaching a year, but during the last year particularly costs had risen at an alarming rate, so that it became extremely difficult to fix a cost per unit. The costs quoted in the

paper were those obtaining at the beginning of 1951, but already they were entirely inapplicable.

With regard to the ventilation of tunnels during the driving of them, he said that for the sake of clarity his pictures had shown the drilling of a hole without water injection. But, of course, water injection was in fact used. It was necessary to cope with the density of gas and dust in the atmosphere in accordance with the regulations, and the densities had been brought down to rather low figures in order to avoid difficulties with the workmen and with the administration.

Discussing Mr. Scott's reference to high head and low head falls, he said M. Brice did not condemn high head—indeed, in France they were constructing high head falls—but his purpose was to point out that there was still a great deal of interest in low head falls. Apart from the scheme dealt with in the paper, quite a number of others of that description were being carried out in France. At one period it was considered that to construct schemes involving a height of less than about 500 ft. for hydro-electric power generation was a waste of time, but nowadays we had come to a more moderate view. His personal impression was that the policy of building enormous plants, such as those in America particularly, would have to be revised ; it was probably more valuable to construct plants of more moderate sizes, and to have more of them. If a plant such as that at the Boulder Dam, for example, were put out of action, those concerned would have a great deal to cope with, whereas if a plant such as that of the Temple Barrage were put out of order, the country would not suffer, because there were quite a number of others in operation.

Of course, Mr. Scott was perfectly right in his advocacy of the provision of reservoirs. Indeed, in France they were tending towards the idea that every hydro-electric installation should have a reservoir, where that was possible. M. de Jarny recalled the film he had shown of the barrage in the Pyrenees and said that its function was precisely that of creating a reservoir for three already existing stations on the run of the river downstream, which previously had suffered the drawback of having too little water on some occasions, whereas on other occasions they had more water than they could use. The reservoir there had a capacity of some 10 million cubic yards of water, and although the scheme was not absolutely ideal from the point of view of regulation, it had effected considerable improvement on the conditions prevailing previously.

There was no question, therefore, that wherever possible reservoirs should be provided in connection with such schemes ; and in fact that was done. In connection with mountain installations, however, involving very high falls, it was not always possible to do that; either the features of the country did not lend themselves to the provision of reservoirs, or the expense involved would be enormous.

But a new technique was beginning to be applied in France, which consisted of reservoiring a glacier and building a gallery underneath for the collection of water. A plant which was more or less experimental had been constructed under Mont Blanc, on the French side, and at a later date we should be able to determine whether or not the plant was really efficient.

Before he could answer Mr. Scott's question concerning load factors he would need more information from M. Brice.

He fancied that the remark concerning the African rivers and their suitability for low head development was his own, and perhaps he should not have made it. In some of the rivers in France the difference in the flows as between high and low tide was infinitely greater than was indicated by Mr. Scott's figures for one of the African rivers ; in one instance, that of the River Truyere, the flow dropped from 100 cubic metres per second to 2 cubic metres per second—a drop of 98 per cent. That was not uncommon in the mountain rivers ; indeed, they were torrents rather than rivers, and engineers in France were very much concerned about the irregularity of the flow in some of them.

Coming to Mr. Hawthorne's question concerning the gates in the turbine intakes, he said it would be necessary to refer to the text before he could answer clearly and adequately. For the moment, he said there was a chain grid to arrest debris which was floating in the water, and there was a gate for each turbine. He believed, but was not certain, that all controls were duplicated, there being both electrically operated and hand operated controls in case either failed ; that applied not only to the turbines, but to the complete outfit.

The charges used in driving the tunnels were detonated electrically. In one of the pictures he had shown there was a fuse, but that was used when taking the photograph merely for demonstration purposes.

Dealing with Mr. Faulkner Nuttall's comment of stimulating the re-investigation of low head schemes, he said he was not sufficiently familiar with the facts to be able to pass judgment concerning the trends in France, but there was no question that installations which, so far as height was concerned, were regarded as being impossible a few years ago from the point of view of efficiency were now being taken into consideration ; due to the improvement of hydraulic and mechanical equipment, those installations were now capable of giving very good service.

(M. de Jarny intimated that he would reply to the remaining questions in writing.)

At the conclusion of the meeting, the President complemented M. de Jarny on the very able manner in which he had dealt with the discussion, and said he had made a most excellent proxy.

On behalf of the British Section of the Société, Mr. R. C. S. Walters expressed thanks to the President, Council and Secretary of the Institution of Structural Engineers for having permitted the use of the lecture room for the presentation of M. Brice's excellent paper.

Written Reply

(A) Hydro-Electric Installation

(1) The ratio between the average and maximum power may vary considerably, according to the desired aim—production of peak load or current by " the run of the river."

In high head installations which operate frequently at peak load, it is possible to obtain, by use of a storage reservoir, up to 1,000 hours of running per year at a pressure much greater than the average (e.g., the Tignes Barrage).

The " run of the river " installations provide the best results in the majority of cases. At Temple, a certain daily reserve gives a peak load for several hours during the autumn and winter evenings, amounting to 2,000 hours. This figure may even be exceeded and a time of 4,000 hours reached.

(2) As far as installations on very great rivers are concerned, the generated power is always small, relative to that possible; so much so that the possibility of linking up over very great distances, from continent to continent, or the use on the spot, is not possible under present conditions.

As to the flow of big rivers, this is a question of type. It is certain that a large catchment area with gently

sloping banks, reduces appreciably the relative importance of floods, but the climatic conditions through which the river passes, may have a considerable influence. It is only necessary to quote the case of the Nile.

(3) Nowadays, very small heads of the order of 2m. are considered for practical use, producing only a few thousand kW.

These installations would be very simplified, composed of a turbine, an asynchronous motor, serving the general grid, controlled by a central station and functioning automatically.

(4) At the Temple installation, the maximum head of water which can pass over the weir is 12m.

The four sluice gates, when opened, allow of a maximum flow of 4,000 m^3/sec., which corresponds to the maximum attainable, account being made for the alteration in level of the water on the downstream face in time of flood.

(5) There are no valves in front of the turbines. The power regulation is obtained by the rotation of blades of a Kaplan wheel and by the opening of a distributor, which are both controlled by electro-mechanical regulators.

Provision has been made, in case of repair being necessary, for the use of cofferdams, formed by individual beams, which are slid into prepared grooves in the concrete.

These units forming the cofferdam, are carried by a lifting device fixed beyond the cleaning apparatus of the grills. It takes about three-quarters of an hour to put them in place.

If, in consequence of a mechanical breakdown, the distributor and the turbine blades do not function, the turbine itself can run without load for something up to an hour.

The turbine and alternator are designed to run at a speed of 2½ times the normal, without damage, during this period of time.

(6) So far as the question of cost price of installations is concerned, it is difficult to give precise indications when we have to bear in mind continually changing values of money.

However, we might say that for Temple Sur Le Lot, the cost of installation, divided by the number of kWH produced per year, is of the order of

$$\frac{3{,}000 \text{ million}}{80 \text{ million}} = 37 \text{ fr.}$$

This figure is one of the lowest. For big installations, such as those on the lower Rhône, we speak of 80 frs. For high head installations, fed by a lake, and supplying peak load current, a figure varying from 150 to 200 frs. may be reached.

(B) Tunnels

The proposed method for driving tunnels is especially studied for those tunnels of small section, which are by far the most frequent.

It has been necessary to develop a process which permits of sufficient speedy progress in the narrow galleries (an average of 8m. metres a day), so as to obtain acceptable cost prices. However, this technique could also be valuable in the case of driving the advanced galleries for tunnels of large section, when the indifferent quality of the soil does not permit the demolition of the whole face.

When the nature of the soil permits, the driving of the gallery may be made for the whole face, the miners using a moving scaffolding for the driving of the upper portion. If the section is sufficiently large, the removal of the spoil may be made directly by lorries circulating in the gallery.

The maximum possible use of water injection is made, when drilling holes, so as to suppress the dust which is unavoidable.

(2) As a measure of security, a certain time must elapse after the explosion of the charges, before the personnel take up their positions again for drilling.

The miner in charge must always proceed to examine the holes himself, in order to test the bottom of the charges. To do this, a good light at the front of the cutting is necessary. This inspection is particularly necessary in the region of the steppings, because of the position and the possibility of the excavated material or even water filling up the holes that have already been formed.

When the bottom of the holes has been established, it is recommended that a live cartridge be pushed into the hole in contact with the charge, so as to explode it. In order to prevent putting the cartridge in contact with fragments which might ignite it or in contact with the walls at a temperature which is too high, it is specified in France that a period of at least half-an-hour must elapse before the actual putting in place of this charge, and as a further precaution, it is necessary to push a ball of greasy clay to the bottom of the hole before placing the charge.

Supplementary Note on the Discussion

The questions asked during the discussion following the lecture have been answered in the earlier responses by M. Brice. Nevertheless two subsidiary questions remain. These are given below, together with the answers.

(a) In France, has there been any trouble due to the corrosion of the concrete, and what precautions are taken to guard against this corrosion ?

The main cause of corrosion is freezing.

There does not seem to be the perfect method of protection. It would appear, however, that a facing of prefabricated concrete blocks of excellent quality, and well anchored to the body of the barrage by suitable reinforcement, might give satisfactory results, provided the joints are made with a certain flexibility.

Experiments with "guniting" have given some results.

(b) Concerning tunnels, experience in England indicates that accidents have occurred as a result of the premature detonation of the charges during storms in the vicinity of the sites. Does the lecturer know of similar experiences in France, and what precautions are taken there to guard against such accidents ?

It is unfortunately true that while using electric detonators frequently enough accidents have occurred.

The cause has not been exactly explained. It seems that it is necessary not only to guard against the action of electrical discharge during a storm, but also against the phenomenon of condenser capacity between the wires and the ground.

A perfect insulation of the wire inside the detonator itself, is essential.

In any case, the following precautions are recommended :

(1) During a dangerous period, electric charging operations should be suspended, and the workmen should move away from the face of the cutting.

(2) At all times, prevent the trailing of the ends of the detonator wires on the ground, especially if this is damp or inundated, and more than ever from touching the rails.

(3) Set up some form of storm detector (such as a wireless receiving station, or barometric gauge).

A New Approach to the Elastic Analysis of Two Dimensional Rigid Frames

Discussion on Mr. Arthur Bolton's Paper*

Corrigenda

THE STRUCTURAL ENGINEER, Vol. XXX, No. 1, January, 1952.

P. 2, col. 1, penultimate line. For $M_{AB} + M_{BC} = 0$ *read* $M_{BA} + M_{BC} = 0$.

P. 3, Fig. 3. In the left-hand sketch, the moment at "C" should be $\frac{2EI\theta_B}{L}$

P. 10, Fig. 15 (b). Top left-hand sketch, immediately under the words "No Shear," the figures .285 and .285 on the stanchion should be in each case —.285.

Mr. BOLTON, presenting the paper, said he would not dwell on the first part of it, nor attempt to deal with the way in which his concept of a method of analysing rigid frames fitted into already accepted methods. The history of science in general over the last 500 years showed that very often new approaches were not so much due to the better observation of facts, better experimental techniques and so on, but rather to a different viewpoint. He wanted to offer, not just a method of analysing rigid frames, but a new viewpoint; he asked the meeting to consider the structure as a whole, rather than individual members of it.

The CHAIRMAN proposed a vote of thanks to the author and declared the meeting open for discussion.

Discussion

Dr. E. H. BATEMAN (Member) said he had listened to Mr. Bolton with great appreciation, and he had gained a much better understanding of the method from the presentation of the paper than he had been able to do from his reading of it. That was due to some of the examples which Mr. Bolton had added, and which could not be included in the published paper.

Having been looking at Mr. Bolton's paper during the week-end, he was glancing at the headlines of a Sunday newspaper and had seen the statement at the top of one column, "There are several different ways of kicking a football." (F.A. Manual on Coaching.) That had led him to the thought that there were a very large number of different ways of analysing the portal frame which Mr. Bolton had given as his first example. The introduction of new solutions from time to time, therefore, was to be regarded as a perfectly natural phenomenon. The portal frame was a very interesting and friendly little job. There was something unique about it; it displayed almost every method to the best advantage, although perhaps the slope-deflection equations, written in full, did not come down quite so compactly. Still, for perhaps a dozen or so of the various methods available, the portal frame gave a neat solution, and Dr. Bateman was glad that the author had extended his work by the two examples he had given at the end. The reader's first reaction to a new system was to discount its application to the simple rectangular portal and to look for the results of more difficult applications.

Going beyond the portal, it appeared that there were still many workable methods from which to choose. Professor A. L. L. Baker,[1] after outlining about half-a-dozen, stated: "An engineer should adopt the method which he finds to be the easiest to understand." Those who had mastered one or two earlier methods would not be too keen to add to their lists the method put forward by Mr. Bolton, and Dr. Bateman looked to some of the students, to the young men who had just come down from college, to contribute to the discussion and to say what they thought about it. As a member of a "rival firm," he declared his interest in methods he had invented himself; so that he was not an impartial judge!

The interesting remark was made quite recently by Professor J. F. Baker[2] that: "During the past 20 years greater advances have been made in the elastic stress analysis of structures than in any other branch of applied mechanics. . . Now there are virtually no insoluble theoretical problems." Dr. Bateman was inclined to put in a claim for some small responsibility for that advance, for he was one of the few people who had solved the fixed base multiple bay frame with pitched or arched roofs,[3] which was beyond the range of the semi-graphical method of R. G. Robertson,[4] the MacLachlan Lecturer of 1948, but was solved by Beaufoy and Diwan[5] in 1949.

Personally, he felt that Professor J. F. Baker had taken much too narrow a view of applied mechanics, but in any case the Professor could not have been thinking of the slope-deflection equations which Mr. Bolton seemed to take as a standard of reference.

The title of Mr. Bolton's paper was very ambitious— "A New Approach . . ." When engineers were asked to look at a new approach to theory of structures, many of them probably had in mind what Sir Richard Southwell said at Bristol in 1949.[6] Referring to the need for new methods of attack on statical problems, he had suggested that inspiration be drawn from Rayleigh's principle, and had said: "To me it seems that its most important feature is in its treatment of the system as a whole—the fact that it deals throughout with integral quantities (the total potential and kinetic energies)."

* *Presented at a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1., on Thursday, January 24th, 1952. Mr. Walter C. Andrews, O.B.E., M.I.C.E. (President) in the Chair. Published in* THE STRUCTURAL ENGINEER *Vol.* XXX, *No.*1, *p.* 1.

References

[1]A. L. L. Baker. "Reinforced Concrete." Concrete Publications, 1949.

[2]J. F. Baker. "Shortcomings of Structural Analysis." ENGINEERING, Vol. 173, p.57. January, 1952.

[3]E. H. Bateman. "Elastic Stress Analysis of Multi-Bay Single-Storey Frameworks." ENGINEERING, Vol. 172, p. 772.

[4]R. G. Robertson. "Semi-Graphical Integration Applied to the Analysis of Rigid Frames." (MacLachlan Lecture, 1948.) STRUCTURAL ENGINEER, Vol. XXVII, No. 11, November, 1949.

[5]L. A. Beaufoy and A. F. S. Diwan. "Analysis of Continuous Structures by the Stiffness and Factors Methods." Q. J. MECH. APPL. MATH. Sept., 1949.

[6]Sir Richard Southwell. "Colston Society Address." RESEARCH, ENGINEERING STRUCTURES SUPPLEMENT (1949).

Thus, Sir Richard wanted the structure to be treated as a whole. At the moment Dr. Bateman was not prepared to say how far Mr. Bolton had gone in that direction. On the evidence so far presented, judgment must be reserved, and Mr. Bolton's application to problems of greater weight would be watched with interest.

Mr. STANLEY VAUGHAN, (Vice-President), thanked Mr. Bolton very heartily, not only for the written paper, but also for the way in which he had clarified the minds of his hearers by his very clear elucidation of the problems involved in the elastic design of rigid frames. He had made out an excellent case to show that his new and original method had certain very definite advantages in the direction of simplicity over other methods in current use.

The author's method had appealed to him particularly because it did not depend on the solution of abstruse mathematical calculations in which one could not visualise what was happening during the mathematical operations. In the author's method, the need for mathematical calculations was replaced by the drawing up and tabulation of the successive steps to be carried out in any given case. This required careful and logical thought but was distinct from the direct application of a mathematical process. Once the operations to be considered had been tabulated, the introduction of the necessary figures and the development of the solution required only elementary arithmetic.

The author had moreover shown in the examples which he had presented that his method was applicable not only to simple problems of rigid frame design but also to the most complicated problems. Mr. Vaughan congratulated the author on the presentation of his paper, and he recommended the method to the attention of engineers.

Mr. S. K. LISYKA (Associate-Member), asked whether the author had thought about applying his method to structures with varying moments of inertia ; in the examples given in the paper it was assumed that the moment of inertia was constant.

He asked also whether it would be possible to make the method more automatic because, although there were many methods, they were complicated. The moment distribution method was purely automatic ; but in the method as at present described by Mr. Bolton one had to think too much about how to deal with the structure.

Mr. JOHN MASON (Hon. Secretary), congratulated the author on his paper, particularly on the enthusiasm with which he put it forward. By his presentation at the meeting he had made the subject very much clearer than it appeared to be from a reading of the paper. Mr. Mason added that he was particularly interested in this problem, and many years ago he had even managed to persuade the Institution to publish his ideas on the subject. But inasmuch as he had not come in for an "Honourable Mention" by Mr. Bolton, perhaps he would be forgiven for not having prepared a script !

However, the author had presented an interesting idea, when he referred to type solutions. After all, every additional tool that the mathematician had in his bag was of value, because there were always arising problems in the solution of which that particular tool would assist. But Dr. Bateman had summed up the position very well when he had said that the easiest method was obviously that which one knew best, and the method which one knew best was equally obviously that which one had struggled to devise for oneself.

In watching the author produce his solutions it had seemed that he was not doing anything other than to solve simultaneous equations, by taking so much of one line and adding so much of another until the items cancelled out. Apart from the idea of the type solutions, Mr. Mason could not appreciate that the author had departed much from the ordinary solution of simultaneous equations.

In that respect Mr. Mason had always felt very strongly that with slope-deflection which deals with the rotation of a joint, involves but one unknown each time, whereas in the solutions the author had put forward he was dealing all the time with at least two : instead of the rotation at (say) B, he was dealing with the moment on one side of B as well as the moment on the other side of B. That, one felt, was an insuperable disadvantage. He agreed about the convenience of dealing with moments, which many people considered was easier to think of than rotations ; but he had never found any great difficulty himself.

Mr. BOLTON expressed agreement with most that Dr. Bateman had said. But he added that it was absolutely possible to deal with a polygonal roof, for instance, or curved members, so long as we knew the carry-over factor and the equivalent stiffnesses. In some cases it might be easier to obtain these by using some other method such as Moment Area or column analogy. But it was still possible to use the method discussed in the paper, to imagine rotations at one end of an arch, and to calculate only the effect carried over to the other.

In answer to Mr. Vaughan, he said it was very important that we should be able to see at every stage of the calculation what was going on in the structure. The other view might very well be taken, that we started with a set of equations, solved them and obtained the answer, but had no need to worry about what happened in between. He considered it very important, however, that we should know what was going on in between, if only to reduce the chances of error and to be able to find any error quickly. It was not much use, having solved a set of simultaneous equations, if we found there was an error but did not know where it was, and had to go through all the work again. When using type solutions of the form he had discussed, intermediate checks at various points could be made, very often automatically, without stopping to think. If one had made an error in the arithmetic it would be found when one of the joints which should be balanced for instance was not in fact balanced, and then one could go back to the last stage at which a check was made. He found it easier to detect errors by his method than by other methods.

In reply to Mr. Lisyka, he said it was possible to use his method with varying moments of inertia. It might sometimes be better to use other methods to calculate the stiffness and carry-over, but it was possible to calculate the stiffness and carry-over by his method. In fact at Manchester examination questions had been set to find the stiffness and carry-over for such members, and this method had been used by students. By splitting up a section where reasonable changes occurred, one could obtain relatively easily the stiffness and carry-over.

As to the suggestion that the method might be made more automatic, he said there were two choices. We could make the method fully automatic, in which case anybody could use it, but R. C. Reese* had quoted one case where, after 23 cycles of balancing by moment distribution, the problem was still far from solution. If we made the method automatic, so that it could be

Grinter. L. E. Discussion on "Wind Stress Ana'ysis Simplified" Trans. Am. Soc. Civ. Eng., 1934, p. 644.

used without applying any thought, in general we should have a lot of arithmetic to do. Mr. Bolton held, however, that it was worth while having a skilled engineer who could use the method so that he did not become a machine, but used his intelligence and decided whether there was a short cut to the answer. He wished it were possible to have an automatic brief solution, but he did not think it was possible.

Expressing agreement with Mr. Mason that it all amounted to the solution of simultaneous equations, he said he hoped the opening part of the paper showed that basically, for that sort of problem, there was no difference between moment distribution, relaxation, slope-deflection and other methods ; they all hoped to solve simultaneous equations and nobody could deny the fact that the solution, when obtained, was the answer to a set of simultaneous equations. But the important matter was which way we should set about solving those equations. If one wrote down 10 or 15 simultaneous equations and attempted to solve them with a slide rule, the errors were likely to be great. If one solved them by successive approximation or moment distribution or relaxation, the errors in general would not be great. With a computing machine which would give many places as easily as two or three, one could solve the simultaneous equations provided there were not too many. But an ordinary engineer could not rely on having a machine ; and Mr. Bolton admitted that he himself took more time with the computer than he would take to work them out otherwise.

All the calculation throughout had been done using a slide rule. One would find figures such as .4366, said to be obtained with a slide rule. He knew that .4366 did not mean much, except that the answer was somewhere about .436, but it was better to carry the extra figure and discard it at the end rather than worry about eliminating a figure on which one could not rely at every point in the calculation. He considered that the method he had put forward was a better way of solving simultaneous equations than merely writing them down and attempting to solve them.

He regarded it not as a difficulty but as an advantage that the individual moments in the members meeting at a joint could be seen. If the engineer cared to ignore them he could, as for instance in Example 5 by the method of Fig. 16, which in contradiction of Mr. Mason's statement, did use one unknown at each joint.

But if the internal moments were retained, the engineer would be able to use any information he might have of the structure to speed the solution. The method shown in Table 1 for example was possible only because the internal moments were known.

Book Reviews

Reinforced Concrete, by Professor A. L. L. Baker : (Concrete Publications, 1949.) 295 pp., $9\frac{3}{4}$ in. $\times$ $6\frac{1}{2}$ in. 15s.

This book is a valuable addition to the up-to-date and useful books included in the well-known "Concrete Series" books on concrete and cement. It is primarily written for students and engineers with an elementary knowledge of theory of structures and who wish to learn the fundamental theory and practical design of reinforced concrete structures. The first chapter deals with general principles of design and includes such questions as working stresses, cost, etc., together with some beautiful photographs of buildings to illustrate æsthetic principles. Nearly one-third of the book is devoted to statically indeterminate structures in Chapter II, and here Professor Baker deals very ably with most methods of stress analysis, including the normal membrane theory for thin-slab vaults.

The usual theories for the design of reinforced concrete beams and slabs, columns and struts are given in a comprehensive manner while secondary effects due to creep, shrinkage and temperature are discussed separately. A chapter is devoted to prestressed concrete, giving all the necessary theory and data for the design of such beams.

The author gives special attention to design practice, Codes of Practice, costing, etc., and emphasises the precautions to be taken in making concrete and in the erection of concrete structures.

It is easy for any reviewer to find fault with most books and, bearing this in mind, the writer feels that the value of the book to students, learning the theory and practice of reinforced concrete for the first time, would be enhanced if it contained a variety of problems or examples. Also it is doubtful whether creep and plastic yield should be treated as two separate phenomena as in Chapter V.

The book, however, covers an extremely wide field and it may be recommended with confidence to anyone interested in the theory and practice of reinforced concrete.

R. H. E.

Road Curvature and Superelevation, by J. J. Leeming, B.Sc., A.C.G.I., M.I.Struct.E., A.M.I.C.E., M.I.Mun.E. (London : CONTRACTORS' RECORD, 1951). 64 pp., 9 in. $\times$ 6 in. 7s. 6d.

The author has condensed into 64 pages a concise account of his own recommendations for the design of transition curves and superelevation with brief sections on road widening, on curves, unsymmetrical curves and the suspended transition. Several useful nomograms and tables are given at the end and sufficient examples of their use are provided in the text. To implement his theory, he first carried out an exhaustive study of the behaviour of all types of vehicles on several existing road curves. In the design of transition curves and superelevation he deplores the wide use of the "design speed" as the criterion because it has a purely theoretical basis. Instead he suggests that superelevation should be related to the "hands-off speed", i.e., the speed at which a vehicle would steer itself around the curve without the driver needing to apply any force to the wheel. However, the desired "hands-off speed" has to be assumed and general guidance only is given on this point.

On the subject of transition curves, he departs from the common theory that its properties depend largely on the length of the transition and instead uses the radius of the circular arc as the basis, thus avoiding the necessity of fitting in Standard transitions from books of tables. Here the experimental data lend great weight to his deductions. He works from first principles throughout the book and for that reason it will be welcomed by practical engineers.

D. M. O'H.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, April 24th, 1952, at 5.55 p.m. Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership ?

STUDENTS

BENG WEE GIAP, of Singapore.

BEZUIDENHOUT, Willem Jacobus, of Germiston, South Africa.

FINNIS, Roy, of London.

GOODWIN, Harry, of Stirling, Scotland.

HAYMAN, Kenneth Maldwyn, of Manchester.

HONES, Dennis, of London.

JOYCE, Charles Peter, of Johannesburg, South Africa.

TURKINGTON, William Kenneth Somme, of Belfast, Northern Ireland.

WALMSLEY, Joseph Roy, of Bolton, Lancashire.

GRADUATES

ANDREWS, Harold John, of Margate, Kent.

BAILEY, Lawrence, of Scunthorpe, Lincs.

BAKER, Christopher James, of Beckenham, Kent.

BAKER, Gordon Arthur Henry, of London.

BARTAK, Andrzej Jozef Jerzy, of London.

BILLINGTON, Roger Dean, of Nottingham.

BRUCE, Kenneth John, of Scunthorpe, Lincs.

BUCK, Edward Frederick George, of Leigh-on-Sea, Essex.

DEMBINSKI, Maciej Jerzy Jozef, B.Sc.(Eng.) London, of Middlesbrough, Yorkshire.

DHAVALIKAR, Dattatry Ganesh, B.E.(Civil) Bombay, of Abmedabad, India.

FOAKES, Jack, of Cardiff.

GARDNER, John Henry George, of Beckenham, Kent.

HANSEN, John, of Stoneleigh, Surrey.

HAWKINS, Cyril Stanley, of Ilminster, Somerset.

HIGGINSON, Leonard, of Mansfield, Notts.

HINTON, James, B.A., B.A.I. (Dublin), of Addlestone, Surrey.

HUGO, Nicholas Louw, of Parow, Cape Province, South Africa.

LEE CHIEW YUEN, of Singapore.

LEWIS, Hugh Elvet, B.Sc. (Wales), D.I.C., of Stoke Poges, Bucks.

LIM TONG PENG, of Ipoh, Malaya.

LOVELL, Geoffrey Robert Wortley, of London.

MCHALLAM, Henry Stuart, of London.

MANN, Frank Thomas, B.Sc.(Eng.) London, of Hastings, Sussex.

MAYO, George Edward, of London.

MOORES, John Herbert, B.Sc.(Eng.) London, of Portsmouth, Hants.

NEILL, Gordon John, of Umbogintwini, Natal, South Africa.

NEWBOULT, Robert Edward, of Bradford, Yorks.

NICHOLSON, Richard Rhodes, B.Sc.(Eng.) London, of Portsmouth, Hants.

PAXTON, Ian Hamilton, M.A.Cantab., of Glasgow.

PHILLIPS, Kenneth Austin, of London.

POULTON, Victor Thomas, of Ilford, Essex.

PRICE, Selwyn Lionel, B.Sc.(Civil) Rand, of Pretoria, South Africa.

PUGH, Kenneth John, of Bromley, Kent.

RIDLER, John Walter Arthur, of Twickenham, Middlesex

ROELOFSE, Neville du Plessis, of Discovery, Transvaal, South Africa.

SARKAR, Suniti, of London.

SMITH, Desmond Evelyn, B.Sc. St. Andrews, of London.

TAYLOR, John Leonard, of Huddersfield, Yorkshire.

THOMAS, Desmond Jethro, of Bridgend, Glam.

WALLAGE, Peter Cecil, of London.

WEBSTER, Hugh Richard, of Orpington, Kent.

WHITLOCK, Christopher John, of London.

WILLCOX, Leslie George, of Walsham, Norfolk.

WILSON, Geoffrey Peace, of Whitkirk, Leeds.

ASSOCIATE-MEMBERS

BROWN, James Archibald, B.Sc. Glasgow, A.M.I.C.E. of Glasgow.

CHAPMAN, Alan, A.M.I.C.E., A.M.I.Mun.E., of Belfast, Northern Ireland.

COX, Allan Thomas Charles, of Watford, Herts.

DE SOUZA, Clarence Xavier Frederick, of Rangoon, Burma.

FERGUSON, John Alexander, of Holytown, Lanarkshire.

FIFE, Gordon Fraser, of London.

FORBES, William Stewart, B.Sc.(Civil) Edinburgh, A.M.I.C.E., of Carlisle.

FORRESTER, Ernest, of Beckenham, Kent.

GIRITSKY, Serge Nicholas, of Lynwood, California, U.S.A.

HAIGH, Jack, of Salisbury, Southern Rhodesia.

HODGENS, Lionel John, of Gateshead, Co. Durham.

KYTE, James Cecil, of Dudley, Worcs.

LEWIS, Cuthbert Pullen, A.M.I.C.E., of Reading, Berks.

MARSTON, George Howard, B.Sc.(Tech.), Manchester, of Manchester.

MOORE, Marcus, B.Sc.(Civil), Bristol, of Derby.

OLDHAM, Thomas, of Grange-over-Sands, Lancs.

RILEY, Peter Inness, of Finchley.

SLATER, Alfred, of Auckland, New Zealand.

TAYLOR, Peter Whitaker, B.Sc., B.E. Hons. (Civil) N.Z., of London.

TUCKI, Joseph, of Oldham, Lancs.

WARD, John Edward, B.Sc.(Eng.) London, A.M.I.C.E., of Leicester.

WEST, Frank Ernest Sidney, of London.

WEST, Harry Wrigley, of Salford, Lancs.

WILLIAMS, Geoffrey, B.Sc.(Eng.) London, of Nottingham

WYATT, John Alfred, B.Sc.(Tech.) Manchester, of Stockport, Cheshire.

MEMBER

MCNAUGHT, Alfred Stanley, of Glasgow.

TRANSFERS

Students to Graduates

AGUIAR, Fernando Alberto, of Johannesburg, South Africa.

AUSTIN, Michael, of Rickmansworth, Herts.

Bond, Derek, of Bristol.
Chappell, Ronald Charles, of Addlestone, Surrey.
Clark, Owen Gladstone, of Johannesburg, South Africa.
Dilks, Raymond Elder, of Seaton Delaval, Northumberland.
Douglas, Roderick Emmanuel, of Port of Spain, Trinidad, B.W.I.
Gallagher, David William, of Sydney. Australia.
Goh Keng Chew, of Singapore.
Harwood, Frank, of Manchester.
Homerston, Ronald William, of London.
Hopkinson, Arthur Barritt, of North Harrow, Middlesex
King, Michael Dexter, of Cobham, Surrey.
Kwiatek, Zbigniew, of London.
Morritt, John, of Batley, Yorks.
Morton, Eric John, of Birmingham.
Power, Leslie Brian, of London.
Reid, James Ferguson, of Wellington, N.Z.
Reith, Ian Hunter, of London.
Ritchie, John William Carter, of New Deer, Aberdeenshire.
Salamat, Muslim Purwez, of Loughborough, Leics.
Savory, Brian Mowbray, of London.
Swaffield, Geoffrey William, of Wallington, Surrey.
Tear, Raymond George, of Brigg, Lincs.
Thompson, Peter Smithson, of Harrogate, Yorks.
Van Zyl, Dennis Christo, of Benoni, Transvaal, South Africa.
Walker, James, of Scunthorpe, Lincs.
Williams, Robert Raymond, of Liverpool.

Students to Associate-Members

Aubrey, William Harry, of London.
Kaplan, Stanley David, B.Sc.(Eng.) Rand, of London.
Nesbit, John Kennard, of Buckhurst Hill, Essex.

Graduates to Associate-Members

Askew, Harold, of Droylsden, Lancs.
Atherton, James, of London.
Betts, Anthony Charles George, B.Sc.(Eng.) London, A.M.I.C.E., of Portishead, Somerset.
Browne, Patrick Arthur, A.M.I.C.E., of New Malden, Surrey.
Bullock, Dennis Terence, B.Sc.(Civil) Birmingham, A.M.I.C.E., of Kingston, Jamaica, B.W.I.
Burgess, Sidney George, D.F.C., of Sanderstead, Surrey.
Carlile, James Sim, B.Sc.(Eng.) Edinburgh, of Edinburgh.
Cooley, Eric Humphrey, B.A. Cantab., of Ruislip, Middlesex.
Cox, Donald Charles, of Motherwell, Scotland.
Duffy, Thomas Desmond, of Warrington, Lancs.
Erby, John, of Bristol.
Evans, Phillip George, of Ellesmere Port, Cheshire.
Fletcher, John Richard, B.Sc. Wales, A.M.I.C.E., of Walsall, Staffs.
Freeman, Donald James, of Coventry.
French, Norman Robert, of Darlington, Co. Durham.
Frostick, Frank Herbert, of London.
Grace, John Stephenson, of Croydon, Surrey.
Gillooly, Lawrence, B.Sc.(Eng.) Rand, of Pretoria, South Africa.
Hancock, Trevor Gordon, M.A. Cantab., of Cleveleys, Nr. Blackpool, Lancs.
Handson, Enderby Frank, A.M.I.C.E., A.M.I.Mun.E., of Maidenhead, Berks.
Herbert, Robin William, of Johannesburg, South Africa.
Hill, George Norman, of Wellington, New Zealand.
Hodgkinson, Allan, M.Eng., Liverpool, of London.
Hook, John William, of Chaddesden, Derby.
Irani, Jamshed Rustom B.E., of Bahraich, U.P., India.
Jenkins, Ralph Alan Sefton, B.Sc.(Eng.) London, A.C.G.I., of London.
Kemp, Kenneth Oliver, B.Sc.(Eng.) London, of London.
King, Gordon Arthur, of Swinton, Lancs.
Kolczok, Ludwik Jozef, of London.
Lewis, Michael Robin, B.Sc.(Eng.) Rand, of London.
Loxton, Peter Pilkington, B.Sc.(Civil) Cape, A.M.I.C.E., of London.
Lynn, Gordon Thomas, of Cheam, Surrey.
Mandelzweig, Gordon, B.Sc.(Eng.) Rand, of Johannesburg, South Africa.
Martin, Raymond Frank, of Horsham, Sussex.
Mellett, Edward Henry, of London.
Meredith, William James, of Edgware, Middlesex.
Mobbs, Edward Barry, A.M.I.C.E., A.M.I.Mun.E., of Alresford, Hants.
Moncrieff, Malcolm Lawrence Anderson, B.Sc.(Eng.) London, of Southport, Lancs.
Mottershead, Geoffrey, B.Sc.(Tech.) Manchester, of Wilmslow, Nr. Manchester.
Newman, Wolfgang Max, B.Sc.(Eng.) London, D.I.C., of Tumut Pond, New South Wales, Australia.
Oldham, Norman Arthur, A.M.I.Mun.E., of New Malden, Surrey.
Pask, John William, of Scunthorpe, Lincs.
Peacock, John Desmond, B.Sc.(Eng.) London, of St. Albans, Herts.
Pearson, James Leslie, of Johannesburg, South Africa.
Rawson, Henry Keith, of Spondon, Nr. Derby.
Rose, Derrick Dudley, B.Eng.(Hons.), Liverpool, of Liverpool.
Shewring, Richard Alexander, B.Sc.(Eng.) London, of Enfield, Middlesex.
Simms, Peter, of Riddings, Derbyshire.
Tait, Alexander, of Motherwell, Lanarkshire.
Van Gilst, Louis Jacobus, of London.
Want, William Allister, of Brisbane, Australia.
Waterman, Harold Leon, B.Eng. Liverpool, of Twickenham, Middlesex.
Whitehead, John, of Warrington, Lancs.
Woodward, Colin Francis, of London.

Associate Members to Members

Hobbs, Alan Cleveland, B.Sc.(Eng.) London, M.I.C.E., A.I.Mech.E., of Potters Bar, Middlesex.
Laithwaite, William Henry, of Sutton, Surrey.

OBITUARY

The Council regret to announce the deaths of Colonel Benjamin Henry Darby Hurst (Member); Ernest William Butler (Retired Member); Leslie Thomas Joseph Smith (Associate); Geoffrey Malcolm Bent (Graduate).

EXAMINATIONS—JULY, 1952

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on July 15th and 16th, 1952 (Graduateship), and July 17th and 18th (Associate-Membership).

EXAMINATIONS—JANUARY, 1952

The Examinations were held in January, 1952, at the usual centres in Great Britain and overseas at Auckland, Baghdad, Beirut, Belize, Bloemfontein, Bombay, Brisbane, Bulawayo, Calcutta, Capetown, Christchurch (New Zealand), Colombo, Cyprus, Delhi (Aligarh), Dunedin, Durban, Hong Kong, Johannesburg, Khar

toum, Kingston (Jamaica), Kuala Lumpur, Lahore, Los Angeles, Lucknow, Madras, Port of Spain (Trinidad), Rangoon, Salisbury (Southern Rhodesia), Singapore, Sydney, Tel Aviv, Wellington (New Zealand).

One hundred candidates took the Graduateship Examination and 276 the Associate-Membership Examination, making a total of 376. Of these, 74 passed the Graduateship Examination, and 98 the Associate-Membership Examination.

The names of the successful Candidates are :—

Graduateship Examination

Aguiar, Fernando Alberto, Alcock, Bruce Lawrence, Alterman, Israel, Altria, Stephen Arthur, Andrews, Harold John, Atkins, George Frederick, Austin, Michael, Bailey, Lawrence, Baker, Christopher James, Ball, Walter Thomas, Bond, Derek, Booth William Harold, Bruce, Kenneth John, Buck, Edward Frederick George, Clark, Owen Gladstone, Dilks, Raymond Elder, Douglas, Roderick Emmanuel, Duerden, Thomas Brian, Durley, John Edward Charles, Evans, Derek William Morral, Gallagher, David William, Gardner, John Henry George, Giritsky, Serge Nicholas Goh Keng Chew, Hansen, John, Harwood, Frank, Hawkins, Cyril Stanley, Hibbert, Norman Trevor, Hofmann, Gottfried, Homerston, Ronald William, Hopkinson, Arthur Barritt, Howdill, Ian Barker, Hugo, Nicholas Louw, Jones, Malcolm, King, Michael Dexter, Kwiatek, Zbigniew, Lee Chiew Yuen, Lim Tong Peng, Lovell, Geoffrey Robert Wortley, McCadden, Michael, McHallam, Henry Stuart, Mayo, George Edward, Moore, David, Morrell, Donald Beaumont, Morritt, John, Mortimer, George, Morton, Eric John, Murray, Ivan Arthur, Neill, Gordon John, Newboult, Robert Edward, Orrell, Harold, Pugh, Kenneth John, Radder, Theodores Johannes Cornelis, Reid, James Ferguson, Ridler, John Walter Arthur, Ritchie, John William Carter, Roelofse, Neville du Plessis, Rose, Douglas Frederick, Salamat, Muslim Purwez, Savory, Brian Mowbray, Simpson, Frank, Slade, Edward Wallace, Spooner, Leslie Allan, Taylor, John Leonard, Tear, Raymond George, Thompson, Peter Smithson, Van Zyl, Dennis Christo, Walker, James, Webster, Hugh Richard, Whitlock, Christopher John, Wilden, Kenneth Leonard, Willcox, Leslie George, Williams, Robert Raymond, Wilson, Geoffrey Peace.

Associate-Membership Examination

Arnott, Kenneth Harper, Askew, Harold, Atherton, James, Aubrey, William Harry, Bennington, Kenward, Betts, Anthony Charles George, Bone, Arthur, Brett, George Edmund, Brewer, John Joseph, Brown, James Archibald, Browne, Patrick Arthur, Brundritt, Alan, Buchbinder, Michael, Bullock, Dennis Terence, Burgess, Sidney George, Carlile, James Sim, Cazaly, Laurence George, Chapman, Alan, Cox, Allan Thomas Charles, Cox, Donald Charles, Crosier, Thomas Arnold, De Souza, Clarence Xavier Frederick, Duffy, Thomas Desmond, Erby, John, Evans, Phillip George, Evans, Peter Robert, Ferguson, John Alexander, Fife, Gordon Fraser, Fletcher, John Richard, Forbes, William Stewart, Forrester, Ernest, Freeman, Donald James, French, Norman Robert, Frostick, Frank Herbert, Gillooly, Lawrence, Giritsky, Serge Nicholas, Haigh, Jack, Hamilton, John Patrick Kean, Handson, Enderby Frank, Herbert, Robin William, Hill, George Norman, Hodgens, Lionel John, Hodgkinson, Allan, Hook, John William, Irani, Jamshed Rustom, James, John Fraser, Kaplan, Stanley David, Kemp, Kenneth Oliver, Kimber, Basil Richard, King, Gordon Arthur, Kolczok, Ludwik Jozef, Kyte, James Cecil, Lewis, Cuthbert Pullen, Lewis, John Charles, Loxton, Peter Pilkington, Lynn, Gordon Thomas, Maclean, Alexander James, Mandelzweig, Gordon, Marston, George Howard, Martin, Raymond Frank, Mellett, Edward Henry, Meredith, William James, Mobbs, Edward Barry, Moore, Marcus, Mottershead, Geoffrey, Nesbit, John Kennard, Oldham, Norman Arthur, Parkinson, George Alfred, Pask, John William, Peacock, John Desmond, Pearson, James Leslie, Porter, Arthur James, Randall, Alan Langford, Rawson, Henry Keith, Richardson, Ronald Edward, Riley, Peter Inness, Rose, Derrick Dudley, Ryell, John, Jenkins, Ralph Alan Sefton, Shewring, Richard Alexander, Slater, Alfred, Simms, Peter, Sundlo, Nils Asbjorn Speilberg, Tait, Alexander, Taylor, Peter Whitaker, Teller, Otto George, Tucki, Joseph, Turner, Robert William, Van Gilst, Louis Jacobus, Want, William Allister, Ward, John Edward, Washington, John Brian, Waterman, Harold Leon, West, Frank Ernest Sidney, West, Harry Wrigley, Whitehead, John, Williams, Geoffrey, Wyatt, John Alfred.

PRIZE LIST—JANUARY 1952 EXAMINATIONS

The Council have awarded the following prizes in connection with the Examinations held in January, 1952.

Andrews Prize (For the candidate who obtains the highest aggregate of marks in the Associate-Membership Examination, passing in all subjects).

A. Brundritt, of London.

Husband Prize (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper " Theory of Structures (Advanced))."

G. F. Fife, of Palmers Green, London.

Wallace Premium (Senior) (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper "Theory of Structures (Advanced))."

J. K. Nesbit, of Buckhurst Hill.

Wallace Premium (Junior) (For the most successful candidate in the Graduateship Examination, passing in all subjects).

S. N. Giritsky, of Los Angeles.

JUNE MEETING

An Ordinary General Meeting of the Institution, for the election of members only, will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, June 26th, 1952, at 5 p.m.

INSTITUTION PUBLICATION, 6/1952

A revised edition of the Institution's publication, " Scale of Charges for Consulting Structural Engineers," is now available and copies may be obtained from the Secretary, price 1s. 3d. each, post free.

REPRESENTATION

The Council have made the following nominations of members to represent the Institution :

Architects' Registration Council of the United Kingdom and Admission Committee

Lt.-Colonel R. F. Galbraith (Vice-President) (re-appointment).

London Building Acts—Tribunal of Appeal
Lt.-Colonel R. F. Galbraith (Vice-President).
Mr. G. B. R. Pimm (Past-President)—Deputy.
(Re-appointed for a further period of three years).

Engineering Joint Council (Session 1952-53)
The President (ex-officio).
Mr. J. E. Swindlehurst (Past-President).

HONOURS

In offering their sincere congratulations to Professor A. G. Pugsley, on being elected a Fellow of the Royal Society, the Council feel that they are also expressing the good wishes of the Institution.

LONDON GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : C. Allen Browne, 43, Coolgardie Avenue, Highams Park, London, E.4.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 29, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

Hon. Secretary : E. R. Deeley, A.M.I.Struct.E., Arranmoor, Adshead Road, Dudley, Worcs.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : M. H. Evans, B.Sc., 42, Church Hill Road, Handsworth, Birmingham, 20.

NORTHERN COUNTIES BRANCH

Hon. Secretary : Ian MacGregor, M.I.Struct.E., 9 Ellison Place, Newcastle upon Tyne, 1.

NORTHERN IRELAND BRANCH

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

Hon. Secretary : D. G. Drummond, B.Sc.,M.I.Struct.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary : E. W. Howells, A.M.I.Struct.E., c/o Messrs. T. Harding & Sons, Ltd., 10-12, Market Street, Torquay.

WALES AND MONMOUTHSHIRE BRANCH

A joint visit to the Penmaenmawr Welsh Granite Quarries and Llandudno has been arranged with the Midland Counties Branch for Saturday, June 7th.

Hon. Secretary : E. R. Steward, A.M.I.Struct.E., Edrom, Ashleigh Road, Blackpill, Swansea.

WESTERN COUNTIES BRANCH

The Branch Annual Dinner was held at the Royal Hotel, Bristol, on Wednesday, February 20th. Among the guests were Mr. Walter C. Andrews (President of the Institution), Mrs. Andrews, Major R. F. Maitland (Secretary), and Professor Andrew Robertson. Entertainment was provided by the B.C.C. West Country Quartet.

The sixth meeting of the Session was held at the Bristol University on Friday, March 7th, when Mr. J. A. Newton (Student) gave a paper on "The Civil and Structural Engineers' Contribution towards the Reconstruction of Stapleton Road Gas Works, Bristol." Professor A. G. Pugsley (Branch Chairman) presided, and a vote of thanks to the Lecturer was proposed by Mr. C. H. Williams (Past Chairman), and seconded by Mr. E. N. Underwood (Vice-Chairman).

A visit to the Stapleton Road Gas Works Reconstruction Site was held on Saturday, March 22nd, when members had an opportunity of inspecting the large-scale works under construction.

Hon. Secretary : C. E. Saunders, M.I.Struct.E., Dunkery, Edward Road, Walton St. Mary, Clevedon, Som.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

Book Review

The Inelastic Behaviour of Engineering Materials and Structures, by Alfred Freudenthal. (New York : Wiley ; London : Chapman & Hall, 1950). 587 pp., $8\frac{1}{2}$ in. $\times$ $5\frac{1}{2}$ in. Price 60s.

This book presents an analysis of a wide range of engineering materials on a basis that is not usually considered by many structural engineers, as it commences with a detailed examination of the intermolecular forces. The book is concerned with problems of research into the mechanical properties of materials and of their behaviour in the testing machine or in structural or machine parts.

The physical and mechanical properties, work hardening, creep and the general elastic and inelastic behaviour of the materials are related to the molecular structure and formulæ are developed for small and large-scale effects.

The reader cannot fail to be impressed by the tremendous amount of work that must have gone into the preparation of this monumental volume. The author has made a very considerable contribution towards the detailed understanding of many of the phenomena associated with widely differing types of materials. Due to the unusual treatment, the book requires careful and detailed study and is likely to have more appeal to the research engineer than to the orthodox designer.

The sections on plastic bending of beams are particularly interesting as they suggest methods of calculating for some of the more complex items associated with this subject which is now commanding so much attention.

W. B. S.

Design of a Paper Store

By E. F. Whitlam, M.Sc.(Eng.), A.M.I.C.E., A.M.I.Struct.E.

Preliminary

Theoretical and practical considerations are, very often, poles apart. It is important therefore that the engineer should be alert, at all times, to offer evidence to show that his designs are behaving in a manner similar to that assumed when the calculations and detail drawings were being prepared. This article describes the structure of a paper store constructed at Watford for Messrs. Sun Printers, Ltd. The main item of interest in the design is that of the northlight portal frames. The various moments in these were calculated by strain energy methods. The movement of one of the frame legs was computed and deflection tests were carried out by a simple method, during construction. The results obtained were found to be in reasonable agreement with the theoretical considerations.

As the calculation of any deflection involves the modulus of elasticity it was a useful opportunity to give practical consideration to this quantity on which there is a lack of information.

Mention is made of some practical considerations when fixing crane rails to reinforced concrete beams ; this is a point which is inclined to be forgotten as it occurs on that border-line between the structural engineer and the plant engineer.

General

One of the initial problems was caused by the small quantity of steel which had been allocated before even preliminary designs and calculations had been made. The total amount of mild steel reinforcement available against the authorisation was 38 tons, and it had to cover both the store and a tunnel connecting it with the main printing works. The tunnel passed beneath the Croxley Green branch line of the London Midland Region of British Railways. Although of orthodox design the tunnel accounted for 12 tons of steel, leaving 26 tons for the main design. The size of the building was 206 ft. 8 in. $\times$ 90 ft. overall and as the tonnage represents a figure of $3\frac{1}{8}$ lb. per square foot of plan area it will be seen that the steel allowance was low.

Fig. 1. Front elevation of paper store

The initial function of the building was for the storage of reels of newsprint and flat sheets of various other grades of paper for photogravure work. A further condition was that the design was to be based on the assumption that at a future date the upper part of the building might be converted into a workshop, and loadings on columns and foundations had to take account of this. Accordingly, a design with north-lighting was chosen so that maximum head-room could be gained for the future floor.

The basic design was for frames at 14 ft. 8 in. centres, two frames spanning across the width of the building. Columns were provided at 14 ft. 8 in. centres in the external walls but only 29 ft. 4 in. centres in the centre of the building, alternate frames being carried on a central longitudinal beam, which was to serve also as a gantry for $1\frac{1}{2}$-ton capacity cranes. These were electrically operated one to each span and running for the full length of the building. The external walls were

$13\frac{1}{2}$ inches thick with $4\frac{1}{2}$ inches cover to the columns. In the interest of economy the end walls were used to carry the roof, thus saving two sets of frames. Normal framing of beams and columns was provided where necessary in these end walls at openings, etc. The roof covering was asbestos cement sheeting, lined with a fibreboard supported in metal strips to provide insulation against heat loss. It was decided to adopt purlins of precast concrete which could be fixed to the frames and at the same time would give a certain degree of longitudinal stability to the building. Although the function of the building was to be solely that of a store and the longitudinal walls were devoid of any features, a most pleasing effect was achieved by a suitable arrangement of the canopy over the main doors and by correct proportioning of the main front wall which was taken up to cover the ends of the northlight frames. This elevation can be seen in Fig. 1, which shows a general view of the building. To the left of the picture the connecting tunnel is just visible. Fig. 2 shows an internal view taken from the far end of the building at crane beam level.

Fig. 2. Internal view looking from rear of building

Site

A pre-piling survey was made, four bores being taken. The results showed that the ground was quite capable of taking loads on isolated foundations of up to 2 tons per square foot, but there was a weakness in the ground at a depth of some 16 or 20 feet below the surface according to position. This would have been at least 10 ft. below the excavation and in consequence it was decided that a bearing pressure of 2 tons per square foot would be used. Owing to the slope of the site the ground at the far end of the building had to be excavated to a depth of about 10 ft. before main foundation excavation started. The majority of the excavation was in flinty chalk and until the land drains were completed, some difficulty was experienced with surface water drainage into the base excavations.

Structural Design

The calculations for precast purlins did not call for any unusual considerations, although the end shear had to be taken carefully into account in view of the change of section (see Fig. 3). The question of fixing was one of satisfactory working dimensions, as the method used was that of anchoring pairs of purlins into the frames by means of a $\frac{5}{8}$ in. dia. inverted U bar which was then grouted in. Hardwood blocks were cast in the glazing purlins spaced to take the glazing bars. The extreme ridge member was cast *in situ* to act as a positive longitudinal tie.

The main northlight frames were designed as a redundant structure. In view of the fact that there was a considerable change of column section above crane beams it was decided to employ hinges at this level. Details of the hinge can be seen in Fig. 3, and Fig. 4 shows a photograph of the hinge bars. In multiple northlight frames the worst effects of moment are

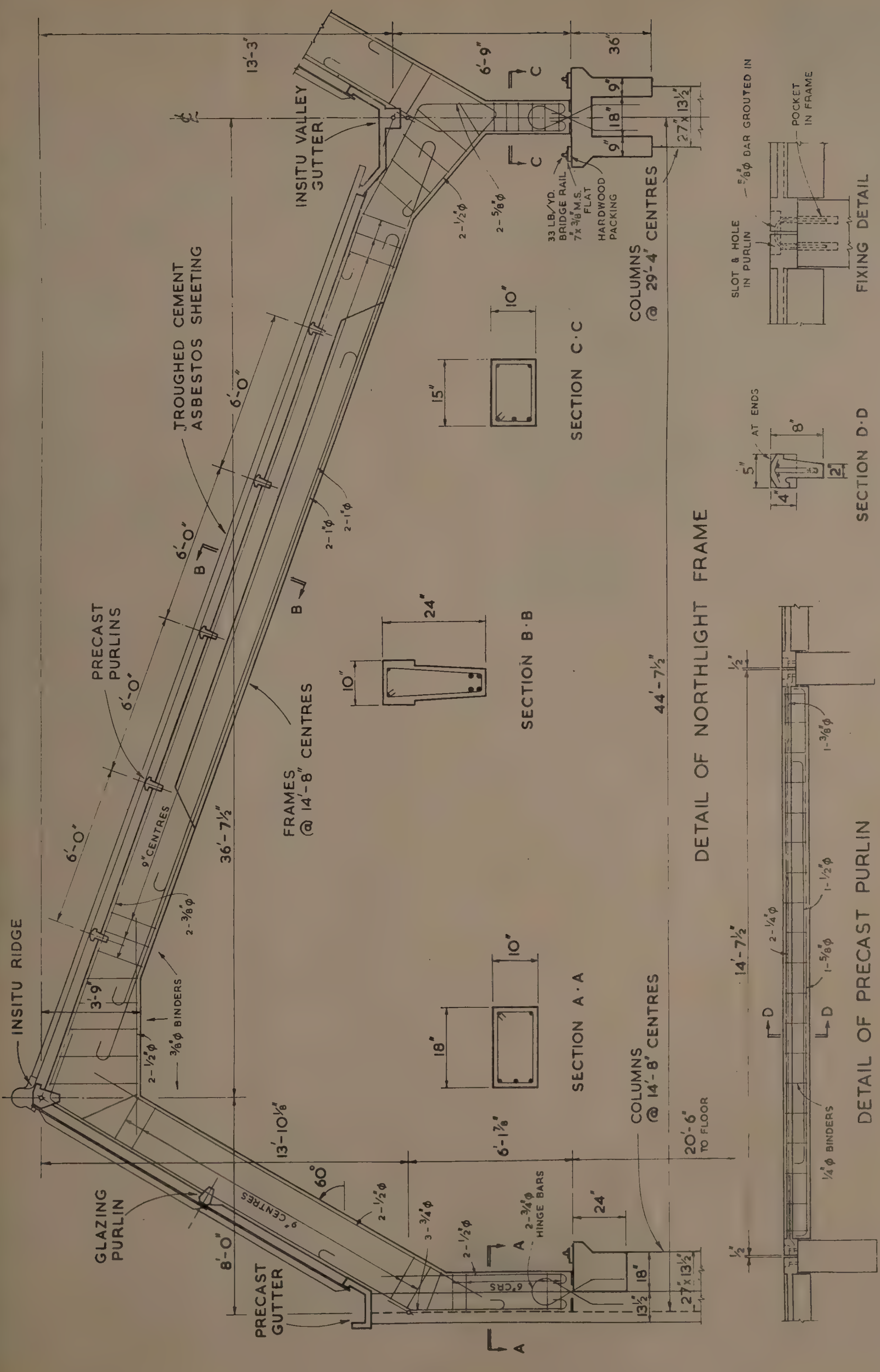
INSITU RIDGE
GLAZING PURLIN
PRECAST GUTTER
PRECAST PURLINS
TROUGHED CEMENT ASBESTOS SHEETING
INSITU VALLEY GUTTER
FRAMES @ 14'-8" CENTRES
9" CENTRES
3/8"φ BINDERS
2-3/4"φ HINGE BARS
6" CRS
60°
COLUMNS @ 14'-8" CENTRES
COLUMNS @ 29'-4" CENTRES
20'-6" TO FLOOR
33 LB/YD. BRIDGE RAIL
7" x 3/8" M.S. FLAT
HARDWOOD PACKING
SECTION A·A
SECTION B·B
SECTION C·C
DETAIL OF NORTHLIGHT FRAME
36'-7½"
44'-7½"
13'-10⅛"
13'-3"
SECTION D·D
AT ENDS
SLOT & HOLE IN PURLIN
5/8"φ DAR GROUTED IN
POCKET IN FRAME
FIXING DETAIL
1/4"φ BINDERS
14'-7½"
DETAIL OF PRECAST PURLIN

Fig. 3.

usually found in the external columns, and in consequence the design was based on a single frame only, the two spans being assumed similar. This arrangement allowed a reduction to be made in the steel in the central column above crane beam level. The use of one frame leads to easier calculation of the various moments, without causing any serious inaccuracy.

A single northlight frame of this form with two hinges has one redundancy, namely, the horizontal

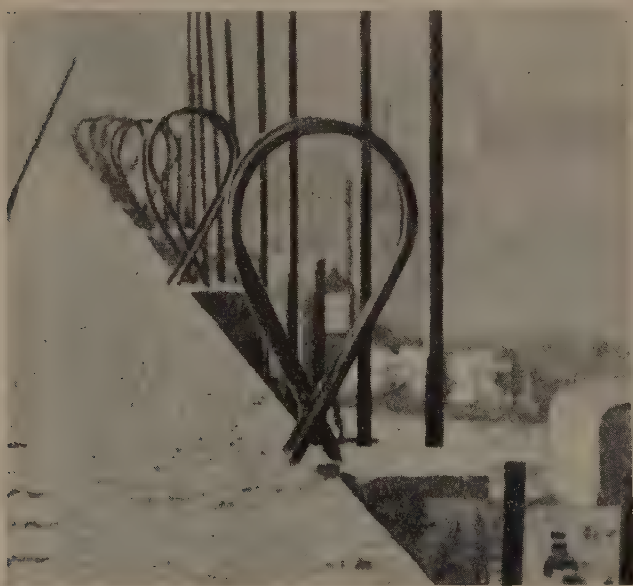

Fig. 4. Hinge Bars

thrust. This can be calculated from the solution of the standard equation.

$$\int \frac{M}{EI} \cdot \frac{\partial M}{\partial H} \cdot ds = \triangle \qquad \text{1}$$

where $\triangle$ = lateral displacement of hinge.
I = moment of inertia of frame.
and ds is measured along the frame.
usually $\triangle = 0$, but in this case it was found

$$\text{from} \quad \triangle \doteqdot \frac{HL^3}{3EI_1} \qquad \text{2}$$

which is the deflection of the column acting as a cantilever and

L = column height
I_1 = moment of inertia of column.

This case arose as there was no lateral tie to offer any horizontal restraint against movement.

The method adopted was the usual one of dividing the frame into suitable increments whence equation 1 becomes

$$\Sigma \frac{M}{EI} \cdot \frac{\partial M}{\partial H} \cdot ds = \triangle \qquad \text{3}$$

and combining equations 2 and 3 the value of H is given from the solution of

$$\Sigma \frac{M}{I} \cdot \frac{\partial M}{\partial H} \cdot ds = \frac{HL^3}{3I_1} \qquad \text{4}$$

Separate solutions were obtained for dead and dead plus snow load, and the equation 4 reduced finally to a linear form

$$aH + b = -cH \qquad \text{5}$$

a, b and c being constants arising from the calculation, the negative sign on the right-hand side indicating the restraint of the column against the lateral thrust.

In the case of wind load, however, it was necessary to obtain solutions of two simultaneous equations arising from equation 4 for (a) dead load and (b) wind load, thus

$$aH + b = -c\ (H - Hw) \qquad \text{6}$$
$$\text{thus} \quad a_1Hw + b_1 = -c\ (H - Hw) \qquad \text{7}$$

a_1 and b_1 being different constants but c remaining as before in equation 5 and H = thrust due to dead load
Hw = thrust due to wind load

It will be seen that the movement occasioned by the wind load has an effect on the thrust produced by dead load and in consequence an effect on the dead load

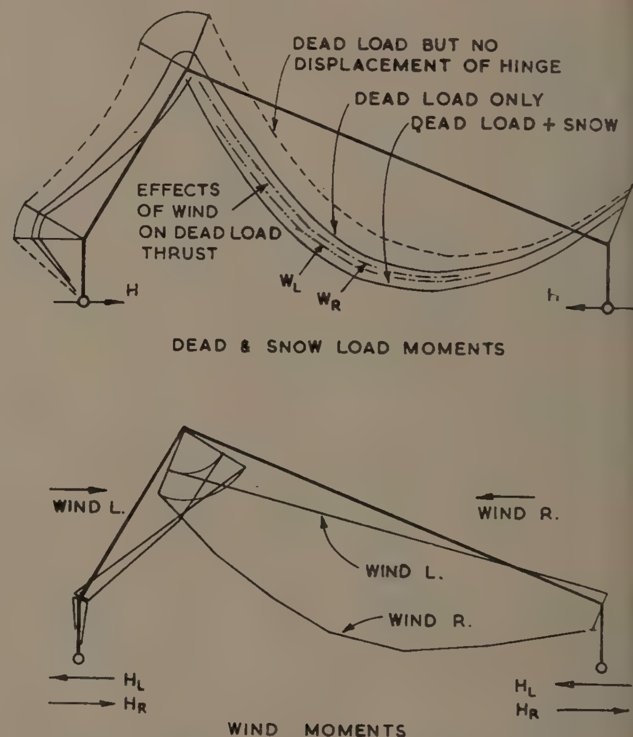

Fig. 5. Bending moments in Northlight frame

moments. The bending moments are given in Fig. 5. The general arrangement of one frame can be seen in Fig. 3.

Crane Beams

The internal crane beams had to take one or other or both of the $1\frac{1}{2}$-ton capacity cranes together with the load of the central northlight frame column occurring at midspan. With this arrangement the maximum bending moments occurring at midspan and support were almost equal, that at midspan being slightly greater. This permitted a fairly uniform arrangement of reinforcement. The loading from the crane was taken as $2\frac{1}{4}$ tons on each of the pair of running wheels, this being the maximum load with the loaded carriage at that side. One of the most important points of practical concern in the design of crane beams is that of

fixing the crane rail to the beam. With structural steelwork crane rails are bolted direct to the crane beams and any excessive local impact which may occur due to irregularities can be taken up in the steel by local overstressing of the beam. Normally this does not give rise to any particular concern. In reinforced concrete, however, conditions are somewhat different. One difficulty occurs in the actual fixing of crane rails to reinforced concrete and another in the fact that reinforced concrete cannot accommodate impact stresses in the same way as steelwork ; thus it is more necessary to ensure a sound bedding down to the concrete. However smooth the tops of the concrete beams may be finished it is desirable to have a packing material beneath the rail. Suitable materials are "Ledkor" damp course, lead asbestos cloth (as used for boiler packings) or bitumen sheeting. If the surface is not even it is preferable to use in addition a continuous timber strip under the rail ; the effect of this is to distribute the load more evenly on the concrete. It is also important to ensure correct vertical alignment of the rails to prevent hammer impact at the junctions. This, if occurring, can easily give rise to shear forces of three or four times the designed value. In the crane beams in question a timber packing on bitumen damp course material was used and the steel bearing plate was continued beneath the junction between rails, which themselves had machined ends. The actual fixing was by means of rag bolts grouted into pockets formed in the concrete.

General Stability

Although the columns were tied continuously at crane beam level it was thought that some form of lateral bracing should be provided to the frames. A continuous *in situ* ridge purlin provided a longitudinal tie to the frames and an *in situ* valley gutter a further tie in the centre of the building. The precast purlins were themselves tied longitudinally but to avoid racking of frames *in situ* slab panels were cast in the penultimate frames. Each panel (there were four of them) spanned the 14 ft. 8 in. between frames and took the place of five purlins on the sheeted slope.

Owing to the considerable dead load coming on to the columns it was possible to accommodate the bending stresses produced by the horizontal thrust without having to provide tension reinforcement as the column section used was always in compression. This resulted in nominal reinforcement and effected considerable economy. The bases were made of such size that the only reinforcement needed was High Tensile mesh fabric and all moments could be fully transmitted through them.

Ground Floor

The loading for this type of floor is 5 cwt. per square foot. The slab was cast on the solid and as the whole of the site had been excavated no settlement difficulty was anticipated.

A 7 in. thick slab was provided on 3 in. blinding. The slab was reinforced top and bottom with High Tensile mesh fabric (4.32 lb. per square yard). The fabric was continued through at normal construction joints. The positions of all the joints were carefully detailed and at the edge of the building and on the line of the central columns a movement joint of a ½ in. bitumen impregnated fibreboard was provided. The object of this was to prevent any settlement on the line of the ground beams under causing cracking. The construction bays of the granolithic finish followed the lines of construction joints in the slab below and the size of bay was limited to approximately 150 square feet. Although there have been slight signs of shrinkage cracks along the construction joints no other floor cracks have been seen in the 21 months the floor has been in use.

Lateral Displacement of Frames

In the calculations the value of the lateral displacement of the external columns was estimated to be between ⅜ in. and ½ in. This is no great amount, but in view of the fact that the crane beams would require to be fixed at accurate centres it was thought that further attention should be given to the question of the actual movement that took place.

Four external columns were selected at the ends of two consecutive frames. It was not convenient, unfortunately, to take more frames owing to the obstruction of the contractors' scaffolding. Of the four selected, two had a central column and two had the northlights at midspan of a main crane beam. The proximity of scaffolding to the external crane beam made it impossible to plumb the columns from this level and so a point 3 ft. below the beam was selected as the upper point and 2 ft. 6 in. above the floor as the lower point. The offset from two scribed lines one at each level on the column was measured by using a plumb bob on very fine twine, suspended from small brackets at the higher level. The measuring was carried out with a steel rule graded in inches and hundredths. It was found that readings could be taken accurately to 1/100 inch, and this was considered to be sufficiently correct for this purpose as it represented about 2-2½ per cent. of the total movement. The difference between the upper and lower readings on the columns was taken to ascertain the "out of plumb." Three sets of readings were taken. The first was taken as soon as the columns and crane beams had been completed and gave the zero error, due note being taken of the direction of this (i.e., whether inwards or outwards) ; it was found that there was only a slight degree

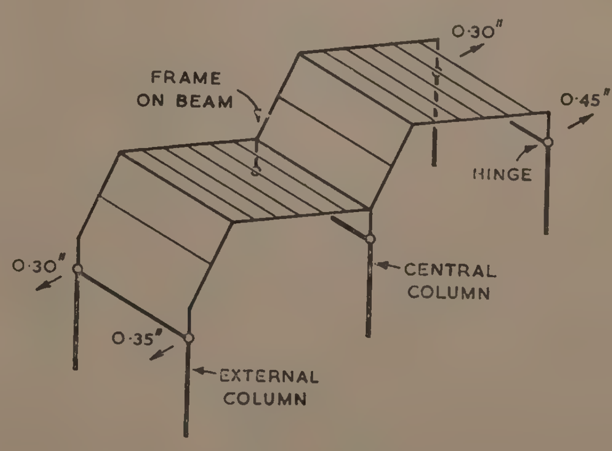

Fig. 6. Values of deflections measured on site

of out of plumb in each of the four columns, the worst case being about ⅛ in. The second set of readings was taken as soon as the soffite shutters of the frames and the propping had been removed. Although the deflection at this stage had not been calculated it was thought useful to record it to show if any excessive initial movement was taking place. The actual values at this stage were from ⅛ in. to 1/5 in., which seemed reasonable.

The final set of readings was taken when the main structural work was complete and the services (heating and electrical) had been installed. These results were not greatly in excess of the second set, being between 1/5 in. and 3/10 in., an increase of some 50 per cent. The values were adjusted in accordance with the deflection curve for a cantilever and were as shown in Fig. 6.

The ratio of the dead load of the frame to the total load of the roof is 1 : 1.7. The ratio of deflections for these cases was 1 : 1.5. The greater initial deflection is probably due to slight creep in the concrete and possibly due to slight movement as the structure took up

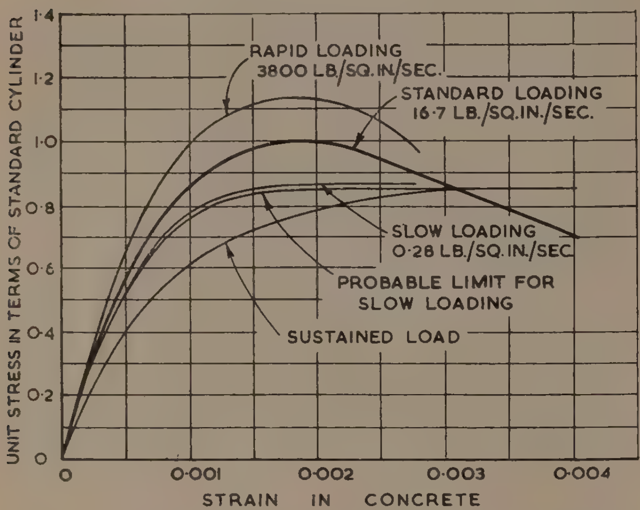

Fig. 7. Whitney's curves for stress—strain relations in concrete cylinders

the load, or to slight resistance of the $4\frac{1}{2}$ inch brick skin (which was only in lime mortar) at the start, increasing as movement took place. The latter may be a preferable explanation as creep should have taken place in both cases. The age of the columns had not increased greatly between the times of second and third readings. The value of the calculated deflection is dependent on the value of E, the elastic modulus for concrete. This is often assumed at 2,000,000 lb. per square inch, or in recent times 3,000,000 lb. per square inch.

An alteration of 50 per cent. in this value would have a corresponding effect on the calculated values of deflections.

The author considers that a more satisfactory value for the concrete actually used on site can be obtained from the cube results. It is fully appreciated that the cube results do not necessarily record the precise strength of the concrete in a structure but are used as a guide to its relative strength providing the same operator is responsible for making them each time. Cubes are not normally used for compression curves to assess the value of the modulus. This is usually done on cylinders, 6 in. dia. × 12 in. long being the size in general use. The cube strength results at 28 days on this contract varied, but from the results obtained it seemed reasonable to take a general value of 5,000 lb. per sq. in. This was for a mix 1 : $1\frac{1}{2}$: 3, and several cube results were considerably higher.

Professor R. H. Evans[1] has given a relationship between cube and cylinder strength. For a cube strength of 5,000 lb. per sq. in. the corresponding cylinder strength is 85 per cent. or 4,250 lb. per sq. in. This stress is the only one at which the ratio is 85 per cent., although 85 per cent. is the ratio often taken for all stresses. C. S. Whitney[2] gave deformation curves for cylinders at various rates of loadings, shown in Fig. 7. Like Professor Evans, he has indicated that 0.2 per cent. or 0.002 is the limiting strain in concrete. The author has found this to be so in cases of slow loading (about 2.50 lb./sq. inch/sec.) on cylindrical specimens. Professor A. L. L. Baker[3] suggests the use of a strain of 0.002 from which the concrete modulus E can be expressed as 500 U, where U = Ultimate Cylinder Strength. This is represented by the line OA in Fig. 8, which shows Whitney's curve for standard and sustained loading. This is the secant modulus at failure and not at working stresses. If the strain be taken as 25 per cent. of this ultimate value, i.e., 0.05 per cent. or 0.0005, the corresponding stress will be about 40 per cent. or 0.4, and the secant modulus will be 800 U, as shown by OB in Fig. 8.

It is felt that it will give a more accurate value of E which for this structure is about 3,500,000 lb. per sq. in. Using this value, the calculated deflection is 0.34 inch, taking the moment of inertia as that for the concrete section of the column. (The steel is negligible in this case.) This compares reasonably well with the values of 0.45, 0.30, 0.35 and 0.30 in. as given in Fig. 6. The fact that two columns have a smaller deflection may be attributed to variations in the quality of the concrete, the ultimate strength of which may have been higher than 5,000 lb. per sq. in.

The architects were Messrs. Stanley Peach & Partners. The Consulting Engineer was Mr. W. C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E., to whom the author

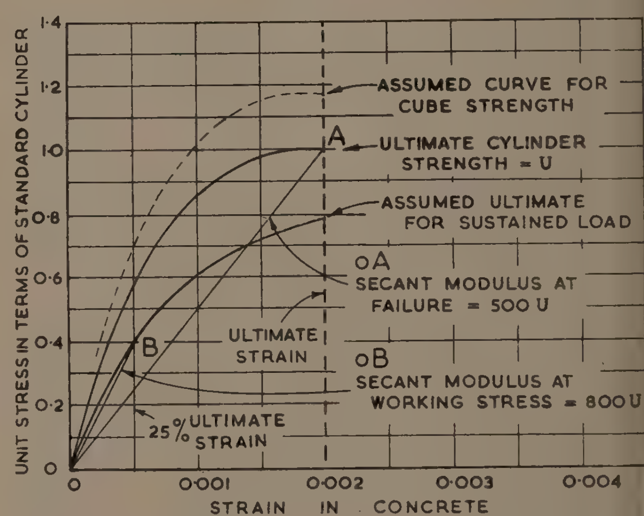

Fig. 8. Suggested value for modulus based on Whitney's curves

expresses thanks for permission to publish this article. The general contractors were Messrs. Wm. Moss & Sons, Ltd.

References

[1]R. H. Evans. "The Plastic Theories for the Ultimate Strength of Reinforced Concrete Beams." J. INST. CIV. ENGRS. Vol. 21 (December, 1943).

[2]C. S. Whitney. "Application of Plastic Theory to the Design of Modern Reinforced Concrete Structures." J. BOSTON SOC. CIV. ENGRS., Vol. XXXV (January, 1948).

[3]A. L. L. Baker. "Recent Research in Reinforced Concrete and its Application to Design." J. INST. CIV. ENGRS., Vol. 35 (February, 1951).

DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH
BUILDING RESEARCH STATION
Note No. E. 336

"Degree of Fixity" Methods for Certain Sway Problems*

By R. H. Wood, Ph.D., B.Sc., A.M.I.C.E., A.M.I.Struct.E., A.M.I.Mech.E. and E. Goodwin, B.Sc.

Summary

"Degree of Fixity" methods have been widely developed for application to those cases of rigid frame analysis where sway may be neglected. In this note, the basis of these methods is examined and it is shown that the concept of "equivalent stiffness" is preferable to that of "degree of fixity." By means of this approach the methods are then extended to solve two problems where sway is of importance, namely, wind loading on multi-storey buildings, and continuous beams on elastic supports.

Introduction

"Degree of Fixity" methods, for the analysis of continuous frames, originated mainly as a development of the Hardy Cross moment distribution technique. Several American writers, notably Evans[1] and Lin[2], contributed to the subject, and were followed by Shepley[3] who produced the treatise best known in this country. In this paper it is proposed to modify and extend these methods so as to be applicable to certain problems involving sway.

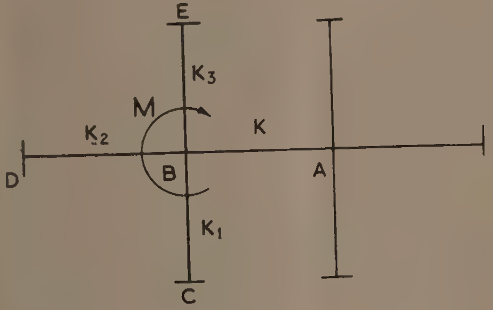

Fig. 1

The "Equivalent Stiffness" Concept

The underlying principle of all such methods may be illustrated with reference to Fig. 1, in which four members of stiffness $\left(\dfrac{I}{l}\right)$ values K, K_1, K_2, K_3 are rigidly connected at the joint B. Let an external moment M be applied to the joint B and consider the moment M_{AB} at the far end of the member AB. The convention of clockwise moments acting on *either* end of a member being positive will be adopted, together with positive clockwise rotations.

It will be found that if $S = K + K_1 + K_2 + K_3$ then :

$$M_{AB} = 2\,EK\left(1 - \frac{K}{4S}\right) 2\theta_A + M\,.\,\frac{K}{2S} \quad . \quad . \quad . \quad (1)$$

Now if the system is replaced by an equivalent beam of stiffness K, encastered at the remote end, then

$$M_{AB} = 2\,EK'\,.\,2\theta_A \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

By comparison of equations (1) and (2) it is evident that the equivalent stiffness of member AB for unit rotation about end A is :—

$$K' = K\left(1 - \frac{K}{4S}\right)$$

The state of affairs may be summarised as follows :—

Moment transmission... $+ \frac{1}{2}\,.\,M.\dfrac{K}{S}$

Stiffness modification ... $- \frac{1}{4}\,.\,K.\dfrac{K}{S}$

The first of these is clearly recognised as the usual Hardy-Cross "carry-over," whilst the second takes the same form but is always a *reduction* to the stiffness. If each of the stiffnesses K_1, K_2, K_3 is in turn modified to K_1', K_2', K_3', according to the true continuity of the frame at C, D, E, then the above expressions become

Moment transmission ... $+ \frac{1}{2}\,.\,M.\dfrac{K}{S'}$

Stiffness modification ... $- \frac{1}{4}$ K. $\dfrac{K}{S'}$

where $S' = K + K_1' + K_2' + K_3'$.

The degree of fixity at the end B of member AB may be expressed as $\left(1 - \dfrac{K}{S'}\right)$, which becomes unity when the member is encastered and zero when it is hinged.

This method of defining fixity is not unique, however, and alternative definitions will be found in references (1) and (3) which lead to different numerical values for any given case. For this reason, and others, the authors consider that "equivalent stiffness" is a more universal concept and it will be used invariably in what follows.

A full exposition of the behaviour of multi-storey building frames, under all critical loading conditions, is given in reference (6), where the "equivalent stiffness" concept has been used to derive a set of simple design charts.

Sway Due to Lateral Loading—"No-Shear" Treatment

The foregoing principles, as they stand, are applicable only in cases where sway may be neglected. Where any lateral loading is present, such as wind, sway is no longer negligible but becomes of some importance.

The solution of this problem, by use of the "equivalent stiffness" concept, will be developed by reference to the simplest possible frame, namely, a symmetrical single bay frame, as in Fig. 2a. It is well known that if fixed end moments are applied to the columns (to prevent the joints rotating due to the wind shears) and then distributed by the Hardy-Cross process, the resulting moments no longer balance the wind shears. Many cycles of moment distribution may be necessary before both moments and shears are balanced. This difficulty may be overcome by observing that, after the first application of the wind loads to the frame, together with the fixed end moments to prevent the joints rotating, the further distribution of the out-of-balance moments must take place in such a manner as to introduce no extra shear to the stanchions. It is now proposed to show how this "no shear" distribution[6] can be linked with the equivalent stiffness concept.

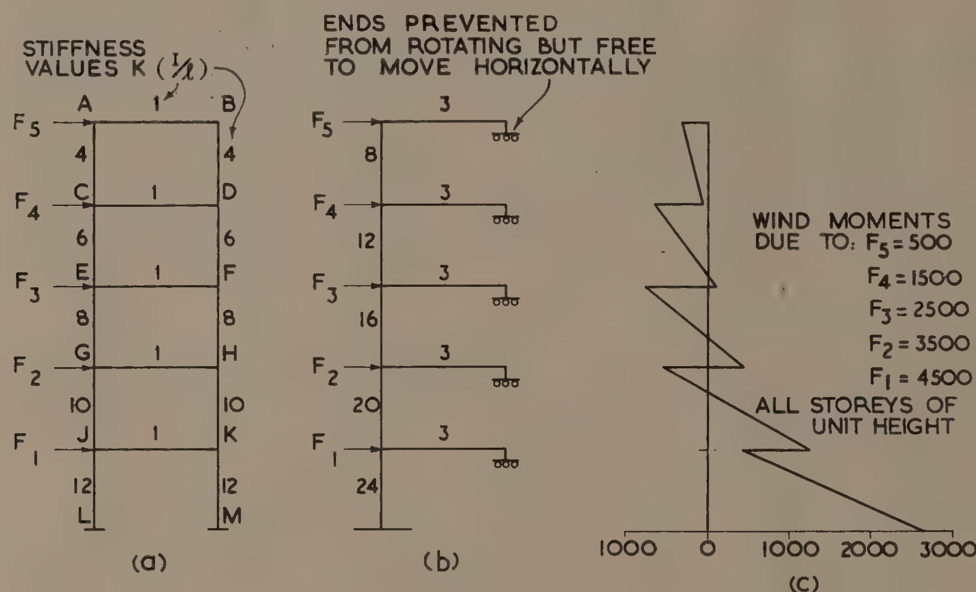

Fig. 2

It will be convenient to replace the frame of Fig. 2a by the equivalent structure of Fig. 2b. The single columns of this imaginary frame have been given the summed stiffnesses of the corresponding real columns, whilst the beam stiffness is arrived at as follows :—

(a) Since the real beams restrain two columns (one at each end) the beam stiffness must be doubled in order to offer an equivalent restraint to the combined column at one end only.

(b) Since the substitute beams have been rotation fixed (for convenience) at their far ends, whereas the real beams are in pure double curvature (central point of contraflexure), a further increase of 50 per cent. is necessary to maintain exact agreement.

The total multiplying factor for the beam stiffness is thus $2 \times 1.5 = 3.0$, and the imaginary frame will develop moments which are the exact *sum* of the corresponding real frame moments.

A part of such a substitute frame is shown generally in Fig. 3, and the theory of "no shear" treatment will now be illustrated.

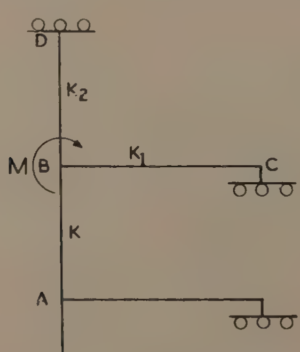

Fig. 3

As before, an external moment M will be applied at B and the resulting moment M_{AB} will be studied.

With the same sign convention as before :—

$$M_{AB} = 2EK\ (2\theta_A + \theta_B - 3R)$$

where R is the sway ($\triangle/h$) of column AB.

Also
$$M_{BA} = 2\ EK\ (2\theta_B + \theta_A - 3R)$$
$$M_{BC} = 2\ EK_1\ (2\theta_B)$$
$$M_{BD} = 2\ EK_2\ (2\theta_B - 3\ R_2)$$
$$M_{BA} + M_{BC} + M_{BD} = M.$$

The sways R and R_2 may be eliminated by applying the condition of no shear in the columns :—

$$M_{AB} + M_{BA} = 2\,EK\,(3\theta_A + 3\theta_B - 6R) = 0$$

$$\text{whence } R = \frac{\theta_A + \theta_B}{2}$$

$$M_{BD} + M_{DB} = 2\,EK_2\,(3\theta_B - 6R_2) = 0$$

$$\text{whence } R_2 = \frac{\theta_B}{2}$$

Then by substituting for R and R_2, the desired result is obtained :—

$$M_{AB} = 2E\,\frac{K}{4}\left(1 - \frac{K/4}{K/4 + K_1 + K_2/4}\right) 2\theta_A$$

$$- M.\,\frac{K/4}{K/4 + K_1 + K_2/4} \quad . \; . \; . \; (3)$$

Stiffness modification $\dfrac{K}{4} \cdot \dfrac{-K/4}{K/4 + K_1 + K_2/4}$

$$= \frac{K}{4}\,\alpha \quad . \; . \; . \; . \; . \; (5)$$

These expressions can be compared with the corresponding ones in the no sway case and the following may be noted :—

(a) The moment and stiffness factors (α) are identical in the "no-shear" case,

(b) The evaluation of the expressions above can be greatly facilitated by a preliminary division of all *column* stiffnesses by four,

(c) The moment transmission corresponds to a "carry-over" factor of -1 (instead of $+\frac{1}{2}$),

(d) In applying equation (4), M can be considered as the applied balancing moment of opposite sign to any out-of-balance moment at a joint. This balancing moment is then split up at the joint in the ratio

$$\frac{K/4}{K/4 + K_1 + K_2/4} \text{ (in effect) and then transmitted}$$

to the far end of the column without change of magnitude but with a further change of sign. The out-of-balance moments themselves may therefore be transmitted without change of sign.

These characteristics, in fact, make a no-shear treatment easier to perform than the no-sway treatment. The division by four automatically allows for the column sways and may be seen to arise from the fact that the

TABLE 1

	Stiffnesses								Moments			
	Column $\Sigma K_C/4$	Beam K_B	S'	α	K'_C	S'	α	K_C'	F.E.M.	DOWN	UP	$\Sigma K'_C + K_B$
A	2.0	3.0	5.00	—.400				1.424	250		920	4.42
					1.200	6.93	—.288		250	350		
C												6.13
		3.0	7.20	—.417				1.934	750		2080	
	3.0											
					1.750	8.43	—.355		750	1210		
E												7.18
		3.0	8.75	—.457				2 434	1250		2990	
	4.0											
					2 170	10.21	—.392		1250	2370		
G												8.38
		3.0	10.17	—.492				3.210	1750		3180	
	5.0											
					2.540	14.00	—.357		1750	3780		
J												11.54
		3.0	11.54	—.520				6.00	2250		2250	
	6.0											
						∞	0		2250	5380		
L												

Comparison of this expression with equation (2) shows that in this case the equivalent stiffness of member AB, for unit *rotation* about end A, is :—

$$K' = \frac{K}{4}\left(1 - \frac{K/4}{K/4 + K_1 + K_2/4}\right)$$

The moment transmission and stiffness modification are summarised below.

Moment transmission... $M. \dfrac{-K/4}{K/4 + K_1 + K_2/4}$

$$= M.\,\alpha \quad . \; . \; . \; . \; . \; (4)$$

rotational stiffness of a free cantilever is $\frac{1}{4}$ that of a propped one.

As the equivalent structure of Fig. 2b possesses no closed rings, the stiffness modifications may be carried to the limit of the structure and an exact solution obtained. The recommended arithmetical process will be best illustrated by treating Fig. 2 as a numerical example, with stiffnesses as shown and wind loads equal to 1000 units per storey ($F_5 = 500$, $F_4 = 1500$, $F_3 = 2500$, etc.), the storeys being of unit height.

The work is set out in Table 1, the arithmetical procedure being as follows :—

(a) Equivalent Stiffnesses

Sum of stiffnesses at $A = S' = 2.0 + 3.0 = 5.0$

$\therefore \alpha_{AC} = -2.0/5.0 = -.400$

$\therefore$Equivalent stiffness of column AC (at end C) $= 2.0\,(1 - .400) = 1.200$*.

Hence at $D : S' = 1.200 + 3.0 + 3.0 = 7.20$

$\therefore \alpha_{CE} = -3.0/7.20 = -.417$

This process is repeated downwards and upwards so that values of K'_c and θ are obtained for each end of each column. Finally, the total *equivalent* stiffness at each joint is entered in the last column. (The beam stiffnesses remain unchanged since their far ends have been rotation fixed).

(b) Moments

The downward transmission from A is :—

$250,\ \alpha_{AC} = 250 \times 0.400 = 100$

(The negative sign of α is, in effect, ignored, as previously explained.)

This transmitted moment is added to the fixed end moment (F.E.M.) at C and a total equivalent F.E.M. of 350 is entered for end C of column AC. This is now added to the F.E.M. below C (740) and similarly transmitted to E,

viz. $(350 + 750) \times .417 + 750 = 1210$.

(Strictly speaking, all the moments should have been entered with negative sign, for the F.E.M. are anticlockwise with a wind blowing left to right.)

The final wind moments are obtained by distributing the equivalent F.E.M. at each joint in proportion to the equivalent stiffnesses, as follows :—

$$M_{AC} = \frac{920}{2}\left(1 - \frac{1.424}{4.42}\right) = 312$$

$$M_{CA} = \frac{350}{2}\left(1 - \frac{1.200}{6.13}\right) - \frac{2080}{2}\left(\frac{1.200}{6.13}\right) = -62$$

$$M_{CE} = \frac{2080}{2}\left(1 - \frac{1.934}{6.13}\right) - \frac{350}{2}\left(\frac{1.934}{6.13}\right) = 658$$

$$M_{EC} = \frac{1210}{2}\left(1 - \frac{1.750}{7.18}\right) - \frac{2990}{2}\left(\frac{1.750}{7.18}\right) = 92$$

$M_{EG} = 785$, $M_{GE} = 465$, $M_{GJ} = 525$, $M_{JG} = 1225$, $M_{JL} = -440$, $M_{LJ} = 2690$.

*It is useful on a slide rule to carry out this particular calculation by setting up such figures as .400 backwards from the right-hand end of the rule.

The moments are halved in order to give the moments in the real frame of Fig. 2a, where the wind load is divided equally between the two stanchions in each storey. The recurrence, in each term of the calculations, of the quantity $K_C'/(\Sigma K_C' + K_B)$ (where K_C' refers to the end of a column for which the moment is being found) is worth noting as an aid to computation.

Remarks on the "No Shear" Treatment

(1) The method of Table 1 is *exact* within the limits of the usual assumptions made in a slope-deflection analysis.

(2) The procedure is "automatic," and the operator does not have to visualise the movements of the structure.

(3) It will be seen that the contraflexure points in the columns are very far from the centre[8], and that the top and bottom storey columns have no such point at all, and are thus in single curvature (Fig. 2c). These effects always occur when the columns are considerably stiffer than the beams. Under such circumstances many well-known methods of estimating wind moments break down. The Hardy Cross process is still valid but becomes very lengthy, since a large number of "cycles" are required.

(4) The method is directly applicable to parallel-chord Vierendeel trusses. For such trusses many different loading arrangements may have to be considered, in which case only the part of Table 1 headed "Moments" is repeated for the alternative loadings.

(5) The rate of convergence in a relaxation attack could be increased considerably by using Naylor's[9] method for symmetrical single bay frames or Bolton's[10] method in more complex frames. The example just considered differs from these methods in the use of the stiffness modification $K(1 - \alpha)$. The authors consider this an important difference, as it leads to a method which is no longer one of successive approximation, the "no-shear" extension of "degree of fixity" methods giving a direct solution for any loading condition.

(6) The method is also valid for single bay frames (and with modifications for other types) where there is a linear slip coefficient for the joints defined by :—

$$\text{Rotation of end } A \text{ of beam } AB = \frac{\sigma}{2EK_{AB}} \cdot M_{AB}$$

By replacing all the beams of stiffness K_B by an effective stiffness $\dfrac{K_B}{1 + 3\sigma}$ an exact solution is still obtained.

Sway in Multi-Bay Rigid Frames

It has been shown by Grinter[7] that any multi-bay frame can be very closely represented by an equivalent frame of the type shown in Fig. 2b. In this case the equivalent column stiffness is the sum of all the column stiffnesses in the corresponding storey in the real frame, whilst the beams are given three times the *sum* of the corresponding real beam stiffnesses. The process is thus a natural extension of the single bay case. However, the imaginary frame is no longer an *exact* equivalent although it is a very close approximation.

It is now necessary to devise a simple method for dividing the column moments in the substitute frame amongst the columns of the real frame. It will be convenient, first, to adopt a ratio Φ, where

$$\Phi = \frac{\text{True wind sway in any storey}}{\text{Sway due to wind if joints were rotation fixed}}$$

Thus for a single column of height h, subjected to a hear F applied at the ends, it is necessary to apply end xing moments of $-Fh/2$ to prevent rotation of the nds. The sway under these (fixed end) conditions will be given by $\delta/h = Fh/12EK$.

Now, if the true wind sway is $R = \triangle/h$

$$\text{Then } \Phi = \frac{\triangle/h}{\delta/h} = \frac{R}{\delta/h} = \frac{12EKR}{Fh} = \frac{-6\,EKR}{-(Fh/_2)}$$

The numerator of this expression is clearly a fixing moment corresponding to the true sway of the column. Hence, if fixed end moments are set up, in each storey of a multi-bay frame, to balance $\Phi \,.\, Fh$ (instead of Fh), the correct result follows from a *no-sway* distribution of these moments by normal Hardy Cross procedure.

The value of Φ for each storey may be calculated by an extension of the process of Table 1. For example, consider the two bay frame with stiffnesses as shown in Fig. 4a. Summing the column stiffnesses, and taking three times the beam stiffnesses, it is seen (Fig. 4b) that we arrive at precisely the same substitute frame as in the previous example. Hence the arithmetic of Table 1 can now be used to illustrate the procedure in a typical multi-bay case.

It can be shown that for any storey of the substitute frame :—

$$\Phi = \frac{12\,E\,\Sigma\,K_C\,\theta a}{Fh} + 1 \quad . \;.\; .\; .\; .\; .\; .\; .\; .\; . \quad (6)$$

where $\Sigma\,Kc$ = sum of column stiffnesses of the real frame and

θa = average of rotations at the top and bottom of the storey in question.

The calculation of θa will be shown with respect to stanchion EG in Table 1 :

$$\left.\begin{aligned} E.\theta_E &= \frac{\Sigma M_E}{4(\Sigma K_C'+K_B)} = \frac{1210+2990}{4\times 7.18} = 146 \\ \text{and similarly } E.\theta_G &= \frac{2370+3180}{4\times 8.38} = 166 \end{aligned}\right\} E\,\theta a = 156$$

Now, from equation (6) $\Phi_{EG} = \dfrac{12\times 156\times 16}{2500} + 1 = 13.0$

Similarly, Φ values can be calculated for all storeys and these, in turn, used to set up F.E.M. on the *real* frame of Fig. 4a, which lead to final wind moments after *one* no-sway Hardy Cross distribution. (The F.E.M. must of course be "weighted" *pro rata* to the column stiffnesses of the real frame).

Since, in the case of multi-bay frames, the treatment is an approximation, it will be found that the final wind moments are slightly out of balance with the wind shears. If extreme accuracy is desired, the out-of-balance shears may be cleared off by a re-application of the whole process. In this case *only the three "Moment" columns of Table 1 need be re-calculated,* followed by a second Hardy Cross distribution with the new set of shears and Φ values.

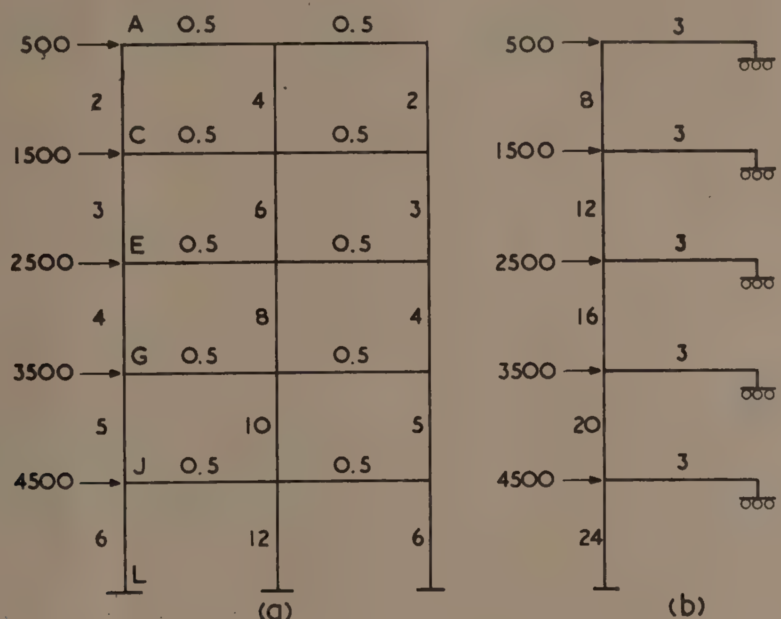

Fig. 4

The important point is that a "no-shear" Grinter frame transforms a sway problem (for multi-storey buildings) into a no-sway problem. When the "no-shear" frame is an exact substitute it is equivalent to an infinite number of Hardy Cross "sway cycles." It is always equivalent, at one operation, to a large number of such cycles. The accuracy of the method is demonstrated in reference (6), where by only one operation a five-storey, two-bay frame was solved to errors of the order of 2 per cent.

Continuous Beams on Elastic Supports

Continuous beams on elastic supports can be solved by a more general application of the principles already studied.

The effective stiffness of such a beam to the right or left of any joint can be illustrated by reference to the equivalent spring supported cantilever of Fig. 5. Consider a rotation θ without any displacement δ. Then M_θ, the resulting moment is equal to $4EK\theta$. Similarly, in Fig. 5(b) with a displacement δ only, it is necessary to supply a downwards force $F = \dfrac{12EK\delta}{L^2} + \epsilon.\delta$ together

with a restraining moment of $M_\delta = \dfrac{-6\,EK\delta}{L}$. Also F_θ in the case (a) is equal to $-\dfrac{6\,EK.\theta}{L}$, which is to be expected from the generalised reciprocal theorem.

There are thus three absolutely independent co-

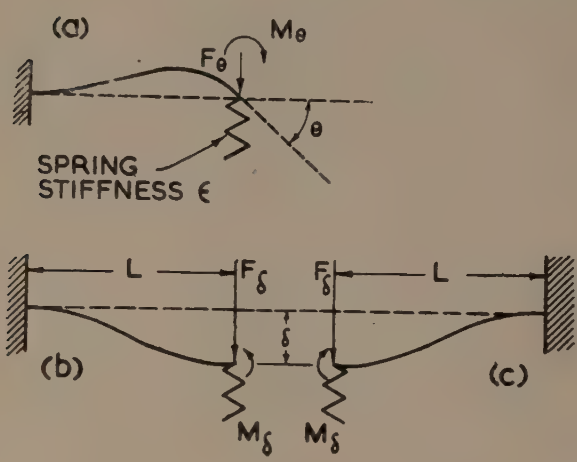

Fig. 5

efficients necessary to describe the system which we designate by

$$m = +\,4EK = \frac{M\theta}{\theta}$$

$$f = +\frac{12EK}{L^2} + \varepsilon = \frac{F\delta}{\delta}$$

and $\mu = \dfrac{-6\,EK}{L}$, being interconnecting term for either $\dfrac{M_\delta}{\delta}$ or $\dfrac{F\theta}{\theta}$. It is essential in this class of work to proceed strictly algebraically. Thus we adopt the convention of clockwise rotations and applied moments $+ve$.

This being the case, when an effective elastically supported cantilever is used to describe the system to the *right* of a joint the coefficients become (Fig. 5c).

$$\left.\begin{aligned} m &= +4EK \\ f &= +\frac{12EK}{L^2} + \varepsilon \\ \mu &= +\frac{6EK}{L} \text{ where } M_\delta = \mu\,.\,\delta\,. \end{aligned}\right\} \quad . \; . \; . \; (7)$$

Note that m and f are always $+ve$, μ is $+ve$ for the left hand end of system 5(c), μ is $-ve$ for the right-hand end of system 4(a). At any joint the left and right-handed systems may be joined together merely by adding the coefficients algebraically together with any additional external torsional restraints (τ) or spring systems (ε) at the joint itself. The combined values of m, f, μ will then describe quite generally any continuous elastic system with one rotation and one displacement in the directions indicated.

Applied moments and forces M and F at the joint would then result in movements given by

$$M = m\theta + \mu\delta$$

$$F = f\delta + \mu\theta$$

from which $\theta = \dfrac{Mf - F\mu}{mf - \mu^2} = \dfrac{Mf - F\mu}{D}$ say

$$\delta = \frac{Fm - M\mu}{mf - \mu^2} = \frac{Fm - M\mu}{D} \text{ say} \quad . \; . \; (8)$$

It is now necessary to derive the transmission coefficients for such a system, and to note in turn how these modify the effective stiffnesses as we proceed from

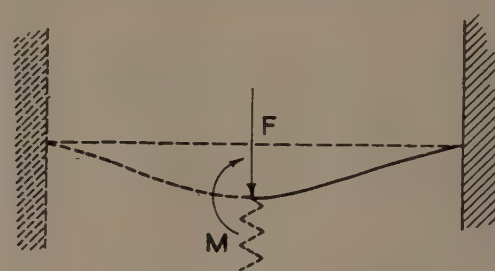

Fig. 6. Release of a Joint: Left to Right

left to right, or *vice versa*. In Fig. 6, suppose that the equivalent system to the left of a joint has been determined together with the equivalent loading for the system, expressed as a F.E.M. *and* a reaction. Now suppose an applied moment M and force F would just "release" the joint including the effects of any loads on the right-hand beam. The problem is to determine what moment and force would be transmitted to the next (right-hand) support. Note that the equivalent system means that *every* joint to the left-hand side is released simultaneously, so that the problem is essentially different from a relaxation procedure. The transmitted moments and forces thus become the new equivalent

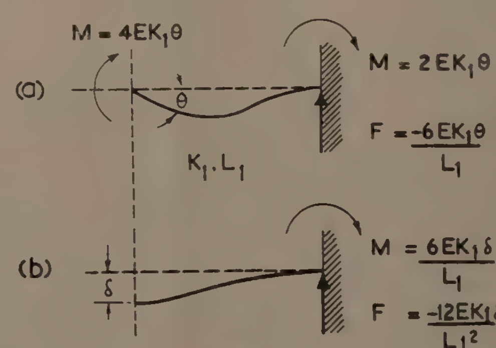

Fig. 7

fixed-end-moments and fixed-end-reactions at the next joint. These must be taken positive clockwise and downwards respectively.

There are four transmission coefficients to study, of which only three are independent. For example, a balancing moment will cause both a fixed end moment

and also a reaction to appear. We therefore designate the transmission coefficients thus

α_{mm}	transmission	coefficient	moment $\rightarrow$ moment
α_{mf}	,,	,,	moment $\rightarrow$ force
α_{fm}	,,	,,	force $\rightarrow$ moment
α_{ff}	,,	,,	force $\rightarrow$ force

In this case α_{mf} is not equal to α_{fm}. Suppose that balancing moments and forces result in movements of θ and δ at the joint in question (Fig. 7). Then with the previous convention these will set up restraining moments and forces at the next joint to the right as follows :—

$$\left.\begin{aligned} M &= 2EK_1\theta = +\frac{m_1}{2}\cdot\theta \\ F &= \frac{-6EK_1\theta}{L_1} = -\mu_1\cdot\theta \end{aligned}\right\}\text{ due to }\theta$$

$$\left.\begin{aligned} M &= \frac{-6EK_1}{L_1}\delta = +\mu_1\,\delta \\ F &= \frac{-12EK_1\delta}{L^2_1} = -f_1\cdot\delta \end{aligned}\right\}\text{ due to }\delta$$

Now let m, f, μ at the joint which is being balanced refer to the combined restraints of all the previous system to the left together with m_1, μ_1, and f_1, and let M and F be the total equivalent fixed ended moments and forces at that joint including the effect of loads on the span L_1. The quantities m, f, μ also include any external torsion or spring systems at the joint itself. Then applying $-M$ and $-F$ as a balance

$$\theta = \frac{-Mf + F\mu}{D} \text{ and } \delta = \frac{-Fm + M\mu}{D}.$$

$$\left.\begin{aligned} &\text{The transmitted moment is } (-M)\,\alpha_{mm} \\ &\text{,, \quad ,, \quad force is } (-M)\,\alpha_{mf} \end{aligned}\right\}\text{ due to a moment balance}$$

$$\left.\begin{aligned} &\text{,, \quad ,, \quad moment is } (-F)\,\alpha_{fm} \\ &\text{,, \quad ,, \quad force is } (-F)\,\alpha_{ff} \end{aligned}\right\}\text{ due to a force balance}$$

Consequently, due to the balancing moment $-M$ there are movements *at that joint* of

$$\theta = \frac{-Mf\delta}{D} = \frac{+M\mu}{D},$$

resulting in a transmitted moment of

$$\frac{-Mf.}{D}\,\frac{m_1}{2} + \frac{M\mu}{D}\cdot\mu_1$$

So that $\alpha_{mm} = \dfrac{f}{D}\cdot\dfrac{m_1}{2} - \dfrac{\mu}{D}\cdot\mu_1$

$$\left.\begin{aligned} \text{and in like manner } \alpha_{ff} &= \frac{-m}{D}\cdot f_1 + \frac{\mu}{D}\cdot\mu_1 \\ \alpha_{mf} &= \frac{-f}{D}\cdot\mu_1 + \frac{\mu}{D}\cdot f_1 \\ \alpha_{fm} &= \frac{+m}{D}\cdot\mu_1 - \frac{\mu}{D}\cdot\frac{m_1}{2} \end{aligned}\right\}\quad .\;\;.\;\;(9)$$

These relationships are also found to hold *without change of sign* for proceeding right to left, providing values of m, f and μ are substituted strictly algebraically. It will be noticed that there are many symmetrical features in the relationships.

It now remains to determine what would be the modified restraint coefficients m'_1, f'_1, μ'_1 at the next joint due to the application of (9) above. If we were to rotate the right hand end of the beam without displacing it, the applied moment would be $m_1\theta$. The moment carried over to the left-hand support, hitherto held clamped, would obviously be $+\frac{1}{2}\cdot m_1\cdot\theta$. There would also be a downwards (positive) restraining force of $-\mu_1\theta$, since we remember that μ_1 for the *right-hand end* of the beam where the modified restraints are required was negative (Fig. 5a). On releasing the left-hand support we transmit back again to the right-hand end a moment of $\alpha_{mm}\,(-\frac{1}{2}\,m_1\theta) + \alpha_{fm}\,(+\mu_1\theta)$ with the right-hand end still rotated through an angle θ.

The final moment at the right-hand support is equal to $m_1\,\theta - \alpha_{mm}\cdot\frac{1}{2}m_1\,\theta + \alpha_{fm}\,\mu_1\theta = m'\theta$ where m' is the required equivalent restraint.

$$\left.\begin{aligned} \text{Hence } m'_1 &= m_1\left\{1 - \frac{\alpha_{mm}}{2} + \alpha_{fm}\cdot\frac{\mu_1}{m_1}\right\} + \tau_1 \\ \text{and similarly } f'_1 &= f_1\left\{1 + \alpha_{ff} - \alpha_{mf}\cdot\frac{\mu_1}{f_1}\right\} + \varepsilon_1 \\ \mu'_1 &= \mu_1\left\{1 - \alpha_{mm} + \alpha_{fm}\cdot\frac{f_1}{\mu_1}\right\} \end{aligned}\right\}\;(10)$$

τ_1 and ε_1 are external torsion and spring systems associated with the supports themselves. It is conceivable that in the general case there could also be an additional term for μ, but this is very unlikely.

In the above formulæ m_1, f_1, μ_1, are to be taken *at the end of the beam* where the modified values are required, and the transmission coefficients at the opposite (previously considered) end. When proceeding left to right μ_1 therefore refers to the right-hand end of a beam and will be negative ; when proceeding right to left, positive. In the above formulæ m_1 and f_1 do *not* include the support characteristics τ_1 and ε_1, which are added separately at the end.

The procedure is to calculate (9) and (10) in turn ; this constitutes one cycle of operations which is repeated at the next joint. In equation (9) m_1, f_1, μ_1 refer to the end of the beam where values of α are required ; in (10) they refer to the end where m'_1, f'_1, μ'_1 are required; this is a convenient and consistent rule to remember.

Before passing on to a worked-out example it is interesting to note that there are two special cases of importance, namely no-sway—which we have already

studied—and the alternative case of a cantilever which is either " free " or given only torsional restraints at the joints. The latter case corresponds to the special value $\varepsilon = O$ and, as we have seen, is of importance in solving problems due to wind loads. In the case of no-sway $\varepsilon = \infty$.

Therefore α_{mm} for no-sway $= \dfrac{f}{D} \cdot \dfrac{m_1}{2}$ and since D tends towards a value mf we find,

$$\alpha_{mm} = \tfrac{1}{2} \cdot \frac{m_1}{m} = \tfrac{1}{2} \cdot \frac{m_1}{m_1 + m'}.$$

The designer can judge the degree of restraint as the calculations proceed, according to the proximity to the limiting values, which is clearly shown in the following example.

Worked Example

A relatively simple example is taken which shows all the salient features. Consider the continuous beam of Fig. 8, with all the various quantities given in consistent tons, ins. units. The values inside the squares refer to the fixed end moments and corresponding

TABLE 2

Left to Right

Point	m'_o	f_o' $x10^{-4}$	μ_o' $x10^{-2}$	m	f $x10^{-4}$	μ $x10^{-2}$	D $x10^{-4}$	α_{mm}	α_{ff}	α_{mf} $x10^{-2}$	α_{fm}
A	∞	∞	∞	∞	∞	∞	—	0	0	0	0
B	4.0	12	—6	9.333	20.65	—0.667	192.1	.3055	—.3635	—.5975	+26.8
C	3.090	1.338	—1.804	9.090	7.376	2.696	59.8	.167	—.480	—.352	+54.8
D	3.030	0.757	—1.283	13.70	16.525	9.387	138.3	—.089	—.682	—.308	+69.[illegible]
E	Not required since end fixed										

TABLE 3

Right to Left

Point	m'_o	f_o' $x10^{-4}$	μ_o' $x10^{-2}$	m	f $x10^{-4}$	μ $x10^{-2}$	D $x10^{-4}$	α_{mm}	α_{ff}	α_{mf} $x10^{-2}$	α_{fm}
E	∞	∞	∞	∞	∞	∞		0	0	0	0
D	10.67	14.23	+10.67	16.67	20.268	6.17	300	.2953	—.3426	+.3966	—31.17
C	3.710	1.174	+1.768	9.043	9.822	—3.565	76.1	.095	—.595	+.357	—50.9
B	2.368	0.981	+1.210	6.368	14.519	—4.79	69.4	.004	—.684	+.427	—41.1
A	Not required.										

For a hinge there is zero restraint $m'_o = O$), and the limiting value of α is 0.5 as in the Hardy Cross procedure.

Similarly, we find that the limiting value of α in a free cantilever is —1. The absolute limits of the coefficients are given as follows, with either free or hinged ends.

	Free Cantilever		No Sway	
	$L \rightarrow R$	$R \rightarrow L$	$L \rightarrow R$	$R \rightarrow L$
α_{mm}	—1	—1	$\frac{1}{2}$	$\frac{1}{2}$
α_{ff}	—1	—1	0	0
α_{mf}	0	0	$-\frac{3}{2L}$	$\frac{3}{2L}$
α_{fm}	$-L$	$-L$	0	0

fixed end reactions negative since acting upwards on the beam.

For convenience we divide all the quantities in the table by $E = 13{,}000$ tons in.^{2}. Commencing at the left-hand end, the transmission coefficients are clearly zero because of the built-in ends. The equivalent restraints to the left of B are therefore the original values of m, f, and μ for AB, taking μ as negative since it refers to the right-hand end of AB (entered under m_o', f_o', μ_o'). The total restraints at the joint are then obtained by adding the values of m_1, f_1, μ_1 for the next member BC. In this case μ_1 will be positive since it refers to the left-hand end of BC.

Thus $\mu = -6 + 5.333 = -0.667 \times 10^{-2}$

$f = 12 + 7.11 + 1.538 = 20.65 \times 10^{-4}$, which includes ε_B.

At B we have $\alpha_{mm} = \dfrac{20.65}{192.1} \cdot \dfrac{5.333}{2}$

$$-\frac{(-.667)(5.333)}{192.1} = 5.305$$

$$_{ff} = \frac{-9.33}{192.1} \cdot 7.11 + \quad \text{(ditto)} \quad = -.3635, \text{ and}$$

o on.

N.B.—It is advantageous to write down at once all he four α terms since several multiplications repeat, and here is an orderly sequence of signs.

similarly $f_1' = 7.11 \times 10^{-4} \left\{1 - .3635 - (.5975) \cdot \frac{(1.50)}{2}\right\} = 1.338 \times 10^{-4}$ to which is added ε in the next cycle.

$$\mu_1' = -5.333 \times 10^{-2}\left\{1 - .3055 - (26.8)\ \frac{(2)}{150}\right\} = -1.804 \times 10^{-2}$$

The moments and forces are then transmitted as in the table below :—

TABLE 4

Point	Left to Right: Forces	Left to Right: Moments	Right to Left: Forces	Right to Left: Moments
A				
			—6.88, −1.283 } —5.597	—413, +1.2 } —411.8
B				
	—8.96	**—192**	**—8.96**, —1.344, +.253 } —10.051	**—192**, —115, +6.7 } —300.3
	—3.255, —1.148, **—1.04** } —5.443	+58.7, +240, **+48** } 346.7	**—1.04**	**+48**
C				
			+.497, —1.713 } —1.216	—155.8, +36.9 } —118.9
	—2.610, +1.220 } —1.39	+298.2, —57.9 } 240.3		
D				
	—5	**—125**	**—5**	**—125**
	—4.37, +.35, **—5** } —9.02	+10.3, +444.7, **+125** } 580	**—5**	**+125**
E				

For continuous beams, with constant moment of nertia between any two supports, the formulæ (9) educe to

$$\left.\begin{aligned} m_1' &= m_1 \left(1 - \frac{\alpha_{mm}}{2} - \left|\alpha_{fm} \cdot \frac{3}{2L}\right|\right) + \tau_1 \\ f_1' &= f_1 \left(1 - |\alpha_{ff}| - \left|\alpha_{mf} \cdot \frac{L}{2}\right|\right) + \varepsilon_1 \\ \mu_1' &= \mu_1 \left(1 - \alpha_{mm} - \left|\alpha_{fm} \cdot \frac{2}{L}\right|\right) \end{aligned}\right\} \quad (10a)$$

where | | signifies the modulus or arithmetic value of the term. Thus all terms are negative whether proceeding left to right or *vice versa*, except α_{mm} which can change sign. This is a useful simplification.

At joint C the value of m_1' for beam BC is entered as m_2' for the next cycle of operations. It is calculated as follows :—

$$m_1' = 5.333 \left\{1 - \frac{.3055}{2} - (26.8)\ \frac{3}{300}\right\} = 3.090$$

The F.E.M.'s and *fixed end* reactions (F.E.R.) are first entered in the table (assuming both no rotation and no displacements), noting that the reactions (—8.96, etc.), are negative since they are opposite to a positive downward displacement. In transmitting moments and forces the general rule to remember is that a $+ve$ value for α would change the sign of the F.E.M. or F.E.R., whereas a $-ve$ value of α would leave the sign unchanged (compare with the "no-shear" technique).

For example at D, with all other joints released, the total reaction is —5, —1.39 = —6.39.

$\alpha_{ff} = -.682$ $\therefore$ transmit force of $-6.39\ (.682) = -4.37$

$\alpha_{fm} = +69.6$ $\therefore$ transmit moment of $+6.39\ (69.6) = 444.7$.

The resulting restraining moments at each end are in this case provided automatically by the figures —411.8 and +580 in. tons. To obtain intermediate support moments we proceed as follows :—

At joint C $\Sigma M = 346.7 - 118.9 = 227.8$

$\Sigma F = -5.443 - 1.216 = -6.659$

$\Sigma m' = 3.090 + 3.710 = 6.800$

$\Sigma f' = 1.338 + 1.174 + 1.538 = 4.050\ (\times 10^{-4}$

taking care to include the spring support only once.

$$\Sigma \mu' = -1.804 + 1.768 = -.036(10^{-2})$$

$$\therefore D = 27.55 \times 10^{-4}$$

The rotation θ is obtained from (7) by applying $-\Sigma M$ and $-\Sigma F$.

$$\text{Thus } \theta = \frac{-(227.8)(4.050) - (6.659)(-.036) \times 10^{+2}}{27.55} =$$

$$-32.55 \left(\times \frac{1}{E} \text{ radians}\right)$$

$$\delta = \frac{(6.659)(6.800) - (-227.8)(-.036) \times 10^{-2}}{27.55 \times 10^{-4}} =$$

$$16400 \left(\times \frac{1}{E} \text{ inches}\right)$$

$$\therefore M_{CB} = +346.7 + 3.090(-32.55) + (-1.804 \times 10^{-2})(16400) = -49.5 \text{ in. tons.}$$

present method there are no simultaneous equations to be solved.

(7) A relaxation procedure would converge very slowly with weak supports for any one particular loading. There is a sagging moment at each internal support in the above example.

Beaufoy and Diwan[4,5] have solved the more general case of continuous structures by attacking the problem from the standpoint of a mathematical analogue to the well-known experimental treatment of unloaded models. Fundamentally the treatment is similar, although the types of coefficients used are different from those usually employed in "degree of fixity" methods. Making use of the "elastic centre" conception, they have shown that considerable further developments are possible, although at the moment no simple treatment of structures with several closed rings appears to be forthcoming.

Acknowledgement

The work described in this paper forms part of the research programme of the Building Research Board

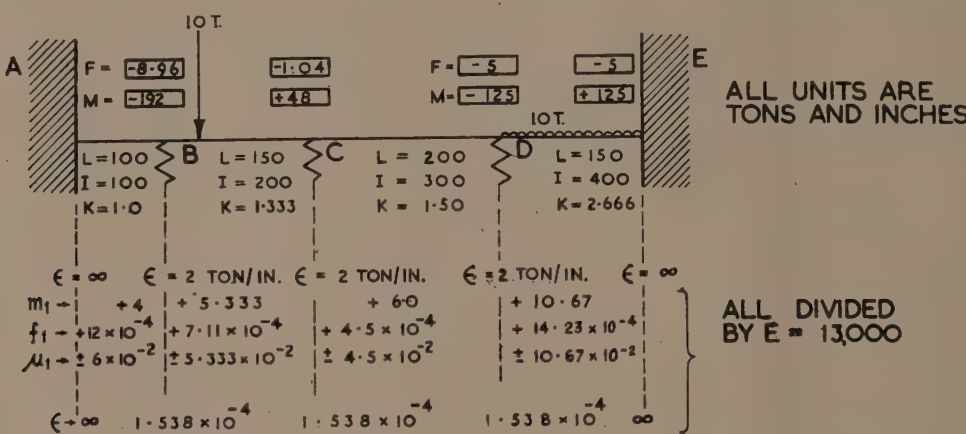

Fig. 8

The complete solution is

$$M_{AB} = +411.8$$
$$M_{BA} = -149.2$$
$$M_{CB} = -49.5$$
$$M_{DC} = -24.5$$
$$M_{ED} = +580$$

The following features are of interest :—

(1) The calculations may be performed throughout on a slide rule. There is no possibility of ill-conditioned equations, subtraction involving large numbers nearly equal, and the like, since we are merely modifying actual restraints from end to end.

(2) Examination of the transmission coefficients gives a mental picture of the proximity to the limiting degrees of restraint at any point.

(3) Influence tables can be made with comparative ease since whatever the load system it is *only necessary to repeat the final table of moments and forces.*

(4) The solutions are "exact" analytically.

(5) As a check on the working, the displacement at B due to unit load at D may be calculated and should agree with the reciprocal effect.

(6) In the orthodox analytical attack a "Five-Moment Theorem" would be required with very complex coefficients to be worked out. To give equal information to the transmission coefficients it would be necessary to work out an Inverse Matrix for the equations. *In the*

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References

[1]Handbook of Rigid Frame Analysis. L. T. Evans, 1934, Edwards Brothers, Ann Arbor, Michigan, U.S.A.

[2]A Direct Method of Moment Distribution. T. Y. Lin. Proceedings of the American Society of Civil Engineers, December, 1934.

[3]Continuous Beam Structures. Eric Shepley, 1942. Concrete Publications, Ltd.

[4]Analysis of Continuous Structures by the Stiffness Factors Method. L. A. Beaufoy and A. F. S. Diwan, QU. JOURNAL APP. MECHANICS, Vol. II, Part 3. Sept., 1949.

[5]Analysis of Pin-jointed Redundant Plane Frameworks using Equivalent Elastic Systems. L. A. Beaufoy and A. F. S. Diwan. QU. JOURNAL APP. MECHANICS. Vol. III, Part 1. March, 1950.

[6]An Economical Design of Rigid Steel Frames for Multi-Storey Buildings. R. H. Wood. H.M. Stationery Office. National Building Studies Research Paper No. 10.

[7]"Theory of Modern Steel Structures." Vol. II. Grinter, Macmillan.

[8]Wind Stresses in Multi-Storey Buildings. M. R. Horne. J. INST. STRUCT. ENGINEERS. June, 1948.

[9]Side Sway in Symmetrical Building Frames. N. Naylor, J. INST. STRUCT. ENGINEERS, April, 1950.

[10]A New Approach to the Elastic Analysis of Two Dimensional Rigid Frames. A. Bolton, J. INST. STRUCT. ENGINEERS, January, 1952.

The Torsional Strength of Structural Members

Discussion on Paper by Dr. W. B. Dobie*

The PRESIDENT proposed a vote of thanks to Dr. Dobie or presenting his paper, and declared the meeting open or discussion.

Professor W. FISHER CASSIE (Hon. Treasurer), first ongratulated Dr. Dobie on having carried the study of orsional strength a little further ; perhaps he had not eached the stage at which the designer could refer to a ook of tables and immediately find the answer, but at east he had taken a step in that direction.

A point which had not been brought out clearly, lthough it was mentioned in the earlier paper, was that his type of investigation in the past, had been made on mall-scale sections or model sections ; work on those mall sections had never proved to be very successful, as ras shown by the variation in the results obtained by ifferent investigators. It was very important to work n a large scale; in the earlier experiments several of he members tested were 24 in. by $7\frac{1}{2}$ in., and he believed he largest sizes were twisted in lengths of 20 ft.

Although the present paper might appear to indicate hat this principle had been abandoned, that was not so, ecause it was shown in the first investigation that the Relaxation methods gave similar results to full scale wisting and to other methods, as pointed out in Reference 2 of the present paper. Full-scale testing, herefore, was really at the back of one's mind in examning the figures.

Perhaps one of the important aspects of the paper vas that concerning plastic design. He asked whether, rom those results, Dr. Dobie had ever considered what e would envisage as the best form of section for members of an I-girder which were subjected to torsion and ending simultaneously. We had the ratio $t_1/c_1 = 1$ s a ratio which gives a strong section, but there were ther optimum ratios which might be devised in a rolled ection subject to torsion and bending.

Finally, Professor Fisher Cassie said how much he had enjoyed reading the paper, and especially examining he diagrams, which could hardly be explained fully in a poken report. He thanked Dr. Dobie for an interesting nd valuable paper.

Mr. A. R. GENT expressed his appreciation of the nvitation to attend the meeting ; the subject under iscussion was of great personal interest to him because e had worked under Dr. Dobie during the last year of he research on which the paper was based, and had been vorking at King's College since. Dr. Dobie had made a lear and concise presentation of a rather difficult ubject.

The paper, in Mr. Gent's view, rounded off the first undamental stage of the torsional investigation, for it overed all the work on ordinary rolled sections subjected o pure torsion. As Professor Fisher Cassie had said, he results could also be used to interpret the behaviour of larger sections and the author had demonstrated that in the paper when he had considered the results obtained by Mackey and Brotton for the torsion constant of a welded girder similar to the type used in the Margam steelworks : he had shown that the increase in the torsion constant which resulted from the welding could be considered as a junction effect. Mr. Gent said that he had since considered other forms of larger structural members and pointed out that for riveted members tests on model sections were of little value and at least semi-full scale members must be used.

Giving a general outline of his work, he said the torsional action of riveted sections was extremely complex because, apart from individual variation, the general behaviour of rivets was non-linear under load, and, therefore, the stiffness curve would be non-linear. The work on riveted sections had been concentrated mostly on explaining the behaviour, as it appeared unlikely that any designer would ever use a method of calculation allowing for a variation of stiffness with load. He had interpreted the results of some full-scale tests on special members fabricated from $\frac{1}{2}$-in. plates and had developed relatively simple formulæ for determining the torsion constant and the maximum stress, which might be of interest as thicker plate was used. Dr. Dobie's ratio of 30/1, between the torque required to exceed the permissible elastic stress in the member and that required to produce the permissible deflection, would, of course, decrease as the thickness of the member increased.

The field of investigation had also been extended to include box and lattice members. Welded box sections presented no difficulty, but the behaviour of riveted ones had been shown by the work of Madsen ("The Iron and Steel Engineer," November, 1941), who had tested members of various types to be dependent on the slip at the seams. The results of ordinary simple shear rivet tests should, however, enable the magnitude of the slip to be anticipated. In lattice members, the problem was complicated as the lattice took part of the shear, thus increasing the torsional strength, and the deformation in the lattice affected the torsional resistance of the flanges. The necessary formulæ had, however, been developed for calculating these effects.

A point which had emerged from his work relevant to the use of the torsion constants, given in the paper, was that the actual stiffness, which is the product of the torsion constant and the modulus of rigidity, is of interest and not merely the torsion constant. Therefore, the accuracy of the values for the torsion constants are wasted unless correspondingly accurate values for the modulus of rigidity are available.

Tests on mild steel plate and bar had led to the conclusion that for mild steel the effective modulus of rigidity was nearer 11×10^6 lb. per sq. in. than the normally accepted value of 12×10^6 lb. per sq. in. That was a difference of 10 per cent. and was greater, he hoped, than anything Dr. Dobie had considered. Mr. Gent did not think the difference could be attributed entirely to the properties of the material and suggested that, to a great extent, it would be caused by the effective

*Presented at a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 28th, 1952, Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXX, No. 2, p. 34.

thickness of the plate being less than the measured value due to the presence of mill scale, and possibly a deterioration of the properties of the material towards the surface. The effective thickness of the plate was very important, as the torsion constant was dependent on its third power, so that a slight error there could give appreciable errors in the results.

It followed, therefore—and that did not appear to have been stressed by previous authors on the subject—that any torsion constant deduced from experimental stiffness tests, using a value of C obtained by the normal methods by measurements of Young's modulus and Poisson's ratio, would be lower than that calculated using the actual external dimensions of the member. This effect probably accounted for the discrepancy between the calculated and experimental results given in the paper for the girder tested by Mackey and Brotton, rather than non-uniform welding as suggested by Dr. Dobie. In general, Mr. Gent's conclusion, from tests on small I-sections and plates up to 1 in. thick, was that the value of 11×10^6 should be used rather than 12×10^6 for the modulus of rigidity of ordinary mild steel.

On a point of detail, he said that Fig. 6 had been used to demonstrate the effect of the re-entrant radius on the stiffness in the preliminary discussion of I-sections. In calculating the curve, however, the flange junction factor α_{JF}, equation 15, had been used, which was not introduced until the work was extended to cover other sections. However, that was only an error in presentation, as the calculated curve in Fig. 6 could also have been obtained from the previous paper, by Cassie and Dobie.

Mr. Gent concluded with a further expression of thanks to Dr. Dobie for his paper.

Dr. E. H. BATEMAN (Member), commenting on Mr. Gent's value of 11×10^6 lb. per sq. in. for the modulus of rigidity, said that for the design of coil springs the recognised value was 11.5×10^6 lb. per sq. in., and he thought that there should not be much difference for mild steel. He invited the author to say what the error would be on a nominal value of 11.5 instead of 12. The 11.5 was obviously an approximation, for there was only one figure of decimals, and the true figure perhaps was somewhere between 11.5 and 11.

Dr. Bateman thought that some of the practical designers present at the meeting must have had their spirits uplifted when they heard Professor Fisher Cassie suggest that a book of tables might be expected ultimately! He hoped that this objective would not be lost sight of.

He went on to ask Dr. Dobie to comment on the problem of bending and torsion combined. Where would the critical point be?

Finally, in a reference to the contribution of torsional strength to the strength of a strut, he said that a strut rarely failed by collapsing in the plane assumed in simple calculation, but generally buckled in torsion as well as laterally. He wondered whether a strut having good strength in torsion would stand up better than one with less torsional stiffness.

Dr. Bateman added his expression of thanks for the interesting way in which the author had presented the paper.

Lt.-Colonel R. F. GALBRAITH (Vice-President), congratulated the author on so interesting a paper dealing with a rather complicated subject, and said that designers owed him gratitude for the work he had done. Table 2, giving the torsional strengths of representative B.S.B. sections, was particularly interesting, and the comparison between the author's figures for T_1 (the maximum torsional strength when the twist was limited to 0.01 radians) and the value T_P (the simple plastic theory strength) was very illuminating. He asked what load factor the author would recommend in using the T_P value.

Dr. DOBIE, replying to the discussion, said he was very pleased to hear that the subject was of some interest to designers. Often he felt that researchers were working in the background and that the results of their labours were not widely used in the design field; certainly they seldom saw the fruits of their labours in material objects. Although the subject of torsion in structures was complicated, he personally found it to be very interesting; for instance, although Relaxation methods appear to be based on an abstruse mathematical concept, they make an absorbing study and they were a powerful tool in many technical calculations.

He was very glad Professor Fisher Cassie had reminded the meeting that their original paper described some large-scale tests, because he considered that such tests were essential. However, they were expensive, and in view of previous work it was thought that numerical computation and other methods were adequate for this study.

The best section to resist torsion had not really been considered, but one of the first things the work had shown was that t_1/c_1 should be about unity. It was not economical to make thick flanges if the web was comparatively thin, or *vice versa*.

Commenting on Mr. Gent's work on riveted sections, he said it was very pleasing that he was considering more practical members. Any mechanism with friction was a nightmare to designers, so that work on riveted sections was most desirable. Non-linearity of the torque-twist curve depended on the transmission of shear by friction across the common surface formed by the two sections which were riveted. The frictional forces might also induce some longitudinal normal stresses if they restricted the warping of initially plane cross-sections.

Referring to the Modulus of Rigidity, he pointed out that the Torsion Constant, which for a circle was equal to the polar moment of inertia, was dependent only on the shape and size of the cross-section; it was not dependent on the Modulus of Rigidity. The results reported by Professor Fisher Cassie and himself were probably accurate enough for design purposes. Here the Modulus of Rigidity of the material was determined by torsion tests on circular cross-section specimens machined from the I-beams after they were tested. Referring to Dr. Bateman's reference to the Shear Modulus for coiled springs, Dr. Dobie said that he thought the elastic constants were not greatly affected by the carbon content, which would be higher than in mild steel. The final decision on the value of the Modulus of Rigidity must however rest with the designer who should know the properties of his building material whether they be mild steel or light alloy.

Regarding Fig. 6, he was indebted to Mr. Gent for pointing out the convenience in calculation which the diagram illustrated. It was quite true that Fig. 6 referred to I-beams, and it was unfortunate that the graph was included in another part of the paper.

With regard to combined torsion and bending of B.S. sections, it seemed to him that the crucial point was likely to be the point of contact with the largest inscribed circle where the radius of curvature was the least. However, each case would have to be considered

separately and the resultant stress tensor would have to be estimated. The shear stresses due to bending and torsion, the normal stress due to bending and possibly the normal stresses due to the restricted warping of the initially plane cross-sections were the stress components. With thick-flanged sections, other points of contact with the inscribed circle might have to be considered and the flange tips might be important if there were much flange bending due to restricted warping.

Torsional buckling of struts might occur at a smaller load than the Euler value if they were more flexible in torsion than in bending. In such cases increased torsional stiffness would give a stronger strut as Dr. Bateman had suggested.

Referring to Colonel Galbraith's request for a load factor on the limiting torque determined by the simple plastic theory, Dr. Dobie felt that the decision must be left to the designer. It would depend mostly on how much deformation could be tolerated ; if one were willing to allow an unlimited angle of twist, one might assume a factor of 2/1 to be safe ; if it were considered that the deformations must be limited it would be better to work with a limiting torque based on calculated deformations, such as T_1.

Finally, in thanking the meeting for the patient hearing accorded him, Dr. Dobie said he hoped the information in the paper could be used in practice where structures must be designed to resist torsion.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, May 22nd, 1952, at 5.55 p.m. Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership ?

Students

BAKER, Derek William, of London.
BOSWELL, Carlton, of Farnworth, Lancs.
CONWAY, Gerald Ernest, of Welling, Kent.
CORSER, Raymond Bidlake, of Twickenham, Middlesex.
MACLACHLAN, Ian Hamilton, of Manchester.
PUN YIN KEUNG, of London.
ROBSON, Keith, of Greenford, Middlesex.
SOWERBY, Paul Leon, of Manchester.
SPEAKMAN, Alan, of Stockport.
WESTON, John, of Liverpool.

Graduates

BADROS, Shawky El-Daba, of Alexandria, Egypt.
CEGLOWSKI, Waldemar, of Manchester.
CONNELLY, Archibald, of Culcheth, Lancs.
FAM, Michael Yue Onn, B.E.(Civil), Western Australia, of Singapore.
GRAINGER, Gerald Park, of Darlington.
HANSON, David Leighton, B.Sc.(Civil) Wales, of Port Talbot.
HEPPLEWHITE, Eric Anthony, B.Sc.(Eng.) London, of Barking.
JOG, Madhusudan Ganesh, B.Sc. Bombay, B.E. Poona, of Poona.
KING, Geoffrey Thomas, A.M.I.Mun.E., of Poole.
KUMARLINGAM, Shankar Nanjappa, B.E.(Civil) Poona, B.Sc. Bombay, of Poona.
NAVIGIRE, Dhondiram Krishna, B.E.(Civil) Poona, of Poona.
NEWBY, Frank, B.A.(Cantab.), of London.
RAMKRISHNA, Hanasoge Suryanarayana Avadhany, B.E.(Civil) Mysore, of Bombay.
RUDRAKSHI, Manmath Sambhuappa, B.E.(Civil) Poona, of Sholapur, India.
RUSSON, Robert George, of St. Pauls Cray, Kent.
SINGH, Kanwar Yaduveer, B.Sc.(Eng.) London, of London.
SPEIRS, Walter Glen, B.Sc.(Civil) Glasgow, of Glasgow.
SUTHERLAND, John Singer, of Stafford.
TRINDER, Stanley, of Middlesbrough, Yorks.
WEEKS, Peter Charles, B.Sc.(Eng.) London, of Plymouth.
WHITTAKER, Denys Beatty, B.Sc.(Tech.) Manchester, of Arkley, Herts.
WILES, Edward, of Manchester.
WRAGG, Walter John Digby, B.Sc.(Eng.) London, of Southend-on-Sea.

Associate-Members

HAMILTON, John Patrick Kean, A.M.I.C.E., of Ruislip.
PARKINSON, George Alfred, B.Sc.(Tech.) Manchester, of Burtonwood, Lancs.

Associate

FULTON, Frederick Sandrock, A.M.I.C.E., of Johannesburg.

Members

MARCUS, Manfred, A.M.I.C.E., of Johannesburg.
QUINTON, Richard John George, of Cape Town.

Transfers

Students to Graduates

BALL, Walter Thomas, of Otago, New Zealand.
BRUNSKILL, Kenneth George, of London.
CREIGHTON, Leslie, of Cramlington, Northumberland.
EVANS, Derek William Morral, of Derby.
FRISCHMANN, Wilem William, of London.
KINDER, Graham, of Stockport, Cheshire.
TANNER, Peter Christopher, of London.
WILDEN, Kenneth Leonard, of Otago, New Zealand.

Graduates to Associate-Members

ALLWOOD, Brian Oliver, of Bolton, Lancs.
ARNOTT, Kenneth Harper, of Derby.
CAZALY, Laurence George, B.Sc.(Civil) Bristol, of London.
JAMES, John Fraser, of Port Talbot.
RANDALL, Alan Langford, of London.
STANDEVEN, Alec, of Manchester.
TURNER, Robert William, of Purley, Surrey.

Associate-Members to Members

KARMAKAR, Taraprasanna, of Singapore.

OBITUARY

The Council regret to announce the deaths of William Edward Higgs (Member) and William Egerton* (Retired Member).

*Founder Member.

ANNUAL GENERAL MEETING

The Annual General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, May 22nd, 1952, at 6 p.m., Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair.

The Secretary (Major R. F. Maitland, O.B.E.), read the notice convening the meeting.

The Minutes of the Annual General Meeting held on May 24th, 1951, as published in THE STRUCTURAL ENGINEER, July 1951, were taken as read and were confirmed and signed.

Mr. E. Granter moved the adoption of the Sessional Report of the Council and the accounts for the financial year, 1951. Mr. F. S. Snow seconded the motion, which was carried unanimously.

Lt.-Col. R. F. Galbraith proposed the re-election of Messrs. James Meston & Co., Chartered Accountants, as Auditors for the ensuing year. Mr. Leslie Turner seconded the motion, which was carried unanimously.

The Secretary then read the report of the Scrutineers on the ballot for the election of the President, the Honorary Officers and the ordinary members of Council for the Session 1952-53, as follows :—

"To the Council of the Institution of Structural Engineers :

Gentlemen,

We, the undersigned, report that at the request of the President, we have duly carried out the duties of Scrutineers of the Ballot for the election of Honorary Officers and Council for the Session 1952-1953, and we report accordingly as follows :

We received 783 Ballot Papers, of which we rejected 22 as wholly spoiled and 13 as partly spoiled. We have attached a separate sheet showing the number of votes received by each candidate.

We declare the result of the Ballot to be as follows :—

Elected

President : Mr. E. Granter, B.Sc.(Eng.), M.I.C.E.

Vice-Presidents : Lt.-Col. R. F. Galbraith, R.E., M.C., B.Sc.(Eng.), A.M.I.C.E.
Dr. S. B. Hamilton, M.Sc., Ph.D., A.R.C.S., M.I.C.E.
Mr. S. Vaughan, B.Sc., M.I.C.E., A.C.G.I.
Professor A. G. Pugsley, O.B.E., D.Sc.(Eng.), M.I.C.E., F.R.S., F.R.Ae.S.
Mr. G. S. McDonald, M.I.C.E.
Professor A. L. L. Baker, B.Sc.(Tech.), M.I.C.E

Honorary Treasurer : Mr. John Mason, B.A.(Cantab.), A.M.I.C.E.

Honorary Secretary : Mr. L. E. Kent, B.Sc.(Eng.), M.I.C.E.

Honorary Librarian : Mr. J. Singleton-Green, M.Sc., A.M.I.C.E., A.M.I.Mech.E.

Honorary Editor : Mr. W. H. Woodcock, F.C.S.

Honorary Curator : Mr. F. R. Bullen, B.Sc.(Eng.), M.I.C.E.

The above are all elected for one year.

Elected as Ordinary Members of Council (London)

Dr. E. H. Bateman, M.A., B.Sc., M.I.C.E., A.M.I.Mech.E.
Mr. P. L. Capper, T.D., M.Sc.(Eng.), A.M.I.C.E.
Mr. F. T. Bunclark, B.Sc., M.I.C.E.

The above are elected for three years.

Elected as Ordinary Members of Council (Country)

Mr. H. E. Manning, B.Sc., M.I.C.E.

The above is elected for three years.

Elected as Associate-Member of Council (London)

Mr. F. M. Bowen, A.I.Mech.E., A.M.I.C.E.

The above is elected for three years.

Elected as Associate-Member of Council (Country)

Mr. W. R. Garrett, A.M.I.C.E.

The above is elected for three years.

We are, Gentlemen,
Yours faithfully,
(signed) C. J. PELL
H. BROMPTON
C. R. GLOVER
H. WINGRAVE NEWELL
(Scrutineers)".

On a motion proposed by the President, a vote of thanks was unanimously passed to the scrutineers.

JULY EXAMINATIONS

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on July 15th and 16th, 1952 (Graduateship), and July 17th and 18th (Associate-Membership).

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in the first category provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

The Colleges which have informed the Institution that courses in Structural Engineering are available are:

- Belfast College of Technology.
- Birmingham College of Technology.
- Bolton Municipal Technical College.
- Bradford Technical College.
- Derby Technical College.
- Dudley and Staffordshire Technical College.
- Glasgow Royal Technical College.
- City of Liverpool College of Building.
- L.C.C. Brixton School of Building, S.W.4.
- L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
- Manchester College of Technology.
- Middlesbrough, Constantine Technical College,
- Salford Royal Technical College.
- South-West Essex Technical College, Walthamstow, E.17.
- Stockport College for Further Education.
- Willesden Technical College, N.W.10.

Colleges in the second category provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete. The colleges which have informed the Institution that such courses are available are:—

- Brighton Technical College.
- Cardiff Technical College.
- Huddersfield Technical College.
- Leeds College of Technology.
- London, Battersea Polytechnic, S.W.11.
- London, Northampton Polytechnic, E.C.1.
- L.C.C. Westminster Technical College, S.W.1.
- Plymouth and Devonport Technical College.
- Preston, Harris Institute.
- Wigan Mining and Technical College.
- Woolwich Polytechnic, S.E.18.

Students attending colleges in the first category are advised to take the organised courses in Structural Engineering. Students of Graduate Membership standard will usually be allowed to select subjects from courses provided by colleges in the second category.

LONDON GRADUATES' AND STUDENTS' SECTION

Hon. Secretary: C. Allen Browne, 43, Coolgardie Avenue, Highams Park, London, E.4.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

Hon. Secretary: A. S. Sinclair, A.M.I.Struct.E., 28, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

Hon. Secretary: E. R. Deeley, A.M.I.Struct.E., Arranmoor, Adshead Road, Dudley, Worcs.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary: M. H. Evans, B.Sc., 42, Church Hill Road, Handsworth, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The Annual General Meeting was held at Newcastle on April 2nd, when the following Honorary Officers and Committee members were elected for the Session 1952-53:—

COMMITTEE—SESSION 1952-1953

Chairman—A. V. Buttress (M)
Vice-Chairman—T. H. Bryce (M)
Immediate Past Chairman—J. Gerrard (M)
Branch Hon. Secretary—Ian MacGregor (M), Messrs. H. Pickup Ltd., Roscoe Street, Scarborough.
Tees Hon. Secretary—O. Lithgow (AM)
Branch Hon. Treasurer—L. Dobson (AM)

Tyne Committee	Tees Committee
E. A. Parsons (M)	W. Fitton (M)
T. L. Usherwood (M)	S. D. Hodgson (AM)
D. W. Cooper (AM)	J. E. Nettleton (AM)
W. H. G. Durose (M)	J. Pringle (M)
C. A. Harding (M)	E. G. Clark (M)
W. R. Garrett (AM)	D. W. Portus (AM)

Hon. Auditors—E. R. Fryer (AM) and E. Atkinson (M)

NORTHERN IRELAND BRANCH

Hon. Secretary: S. G. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The Annual General Meeting of the Scottish Branch was held at Glasgow on April 17th, 1952, when the following Honorary Officers and Committee members were elected for the Session 1952-53:—

Chairman—R. Summers (M).
Vice-Chairman—R. H. Sharpe (M).
Hon. Treasurer—W. Basil Scott (M).
Hon. Secretary—D. G. Drummond (M).
Hon. Auditors—W. Heigh (M) and J. P. Messer (AM).
Committee Members—J. Cameron (M), G. M. Dingwall (M), W. Girvan (AM), D. Grewer (M), H. P. Johnston (AM), A. MacLean (A), H. McClusky (AM), P. Plews (M), A. G. F. Russell (M), W. S. Smith (AM), H. B. Sutherland (AM).

The Immediate Past Chairman is Dr. C. M. Moir (M).

Hon. Secretary: D. G. Drummond, B.Sc., M.I.Struct.E., A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary: E. W. Howells, M.I.Struct.E., c/o Messrs. T. Harding & Sons, Ltd., 10-12, Market Street, Torquay.

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary: G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

Hon. Secretary: C. E. Saunders, M.I.Struct.E., Dunkery, Edward Road, Walton St. Mary, Clevedon, Som.

YORKSHIRE BRANCH

At the Annual General Meeting of the Yorkshire Branch held on Wednesday, April 23rd, 1952, at the Great Northern Hotel, Leeds, at 6.30 p.m., the following were elected to the Committee for the next Session :—

Chairman—D. R. S. Wilson (Member) .
Senior Vice-Chairman—J. Dossor (Member).
Junior Vice-Chairman—L. Preston (Member).
Immediate Past-President—A. Robb (Member).
Hon. Secretary—E. Wrigley (Associate-Member), 17, The Drive, Alwoodley, Leeds.
Hon. Auditors—C. C. Begg (Associate-Member) and Major H. C. Taylor (Associate-Member).

The following were elected as ordinary members of Committee for the next three Sessions :—

James Bussey (Member).
T. F. Cliffe (Member).
H. C. Husband (Member).
H. E. Manning (Member).
J. P. Robinson (Associate-Member).

It was agreed that the Leeds Meetings of the Branch next Session should be held at the University.

After the business of the Branch had been dealt with, a paper on "The South Bank Roof" was given by Mr. S. Woolf, B.Sc., of the Timber Development Association, Ltd., and was illustrated ; an interesting discussion followed. A vote of thanks to the speaker was proposed by Mr. R. Jones (Associate-Member) and Mr. J. P. Robinson (Associate-Member), and warmly accorded.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

Book Reviews

Hardenability and Steel Selection, by Walter Crafts and John L. Lamont. (London : Pitman, 1949. 279 + xiii pp. ,9 in. × 6 in. 35s.)

The last 20 years have formed a period during which great advances have been made in the conception of the relationships between micro-structure and mechanical properties of steels. Experimental and theoretical work on depth hardening characteristics have discovered methods by which all the essential mechanical properties of steels heat treated in any thickness can be estimated with precision which compares favourably with the variations introduced by the ranges of composition. In the U.S.A. the profusion of articles on this subject can be embarrassing, and Crafts and Lamont, who have been in the forefront of workers in this field, have made a welcome contribution to the literature by summarising recent American work. This is presented in readable form together with the basic equilibrium and transformation diagrams and a chapter on steel selection which places due emphasis on economics, the value of published Jominy curves and the judgement which is founded on experience. There are four appendices dealing with steel specifications and hardness conversion tables, adequate indexes and an alphabetical bibliography of 139 references.

The authors state that the least predictable mechanical property of a steel is its resistance to repeated loading and they devote only one paragraph to fatigue strength, which is discussed as "endurance limit," presumably referring to alternating loading. As the book is written principally from the viewpoint of engineers and metallurgists in the field of mechanical engineering, it is perhaps natural that weldability receives only a reference as an important variable and that the partial heat treatment of structural components such as rails is ignored. However, in view of its great economic importance, it is surprising that the hairline cracking of alloy steels is not mentioned in the whole of the 279 pages.

Another strange omission—at least to British metallurgists—is the name of the constituent Sorbite which becomes ferrite with a rejected "transtition structure" of carbide. However, this omission is redressed by the wisdom of the authors in conceding to European readers a free translation into degrees Centigrade of nearly all temperatures cited in the text and shown on the diagrams. G. S. G.

Elements of Soil Mechanics in Theory and Practice, by K. Ll. Nash. (London : Constable, 1951.) Size, 8 in. × 5 in. Pp. 110. 9s.

The book starts with a brief but very informative history of soil mechanics which emphasises the all-important fact that although the term "Soil Mechanics" is new, its methods have been known and applied unconsciously by engineers for many years. They have, however, only recently been correlated and standardised sufficiently to form a recognised method of investigation. The history is followed by brief descriptions of the methods used on site investigation and testing, indicating the general principles involved and omitting details which can only concern the more advanced student. Finally, there are very informative descriptions of the application of the results obtained by soil mechanics methods. These descriptions really do demonstrate how soil mechanics can help the practical engineer.

In connection with the passage on site exploration (pp. 14-21) ; it is considered that more emphasis should be laid upon the importance of careful consideration of the choice of methods used, the extent of the investigation and its accuracy. It is obvious that all the care in the world over testing soil properties and applying the results to engineering calculations will be futile if they are applied to soils not truly representative of those which are subject to stress and other factors imposed by the work to be undertaken. The fact that the exploration is usually undertaken in places remote from the rigid control of the office or laboratory, makes it all the more important to stress the value of careful and expert consideration.

The author is to be congratulated on the immense amount of useful information which has been condensed into so small a book. The student who aims at a practical engineering career will obtain from this book a very good general idea of the methods and application of soil mechanics. To one who aims at specialising, it is an excellent preparation for the more advanced works. The concise and interesting manner in which the book is written will also make it very acceptable reading for the more experienced engineer who has not had the benefit of an academic training in the subject. C. B. B.

Beams on Elastic Foundations— Solution by Relaxation Methods

By W. Wright, B.Sc., Ph D., A.M.I.C.E.

Several pure mathematical solutions have been obtained for beams resting on elastic foundations, see, for example, Terzaghi's "Theoretical Soil Mechanics," but these are restricted to certain standard types of loading and to constant flexural rigidity of the beam.

A practical application of these solutions is represented by a beam (usually concrete) resting on a soil which is assumed to have a constant subgrade modulus; this modulus, being the pressure necessary to deflect the foundation unit distance, has dimensions (WL^{-3}).

In practice, there is a considerable variation in the value of the subgrade modulus, the assumed flexural rigidity of the beam and the assumed values of load. This is, therefore, a type of problem particularly suited to solution by Relaxation Methods[1]; an exact solution is unobtainable but a solution to any required degree of accuracy is easily found. In view of the uncertainty of the other data, particularly the subgrade modulus, the degree of accuracy aimed at need not be exacting.

The relaxational method is readily adaptable to any type of loading and to any variation in flexural rigidity along the beam, whereas such variations tend to make the solution by pure mathematics very complex and tedious.

Governing Equation—

The governing equation is, from the usual beam theory,

$$EI\,\frac{d^4y}{dx^4} + ky - w(x) = 0,$$

where

x, y are Cartesian co-ordinates
EI = the flexural rigidity of the beam
$w(x)$ = the loading intensity at x
k = Subgrade Modulus.

As a test of the relaxation method, the "standard case" of a central point load acting on a beam of uniform flexural rigidity has been worked out and checked against the solution as obtained by the more conventional treatment.

Relaxational Approach

The beam is divided into a number of equal parts "h" apart. The governing equation is then expressed in finite difference form and values of the wanted function y are guessed at each point in the range. These values are substituted in the finite difference equation for the point and the resultant error or residual found. This is then liquidated by means of suitable operations, the final residual being made as small as we please, compatible with the accuracy required. When the deflections have been found, the practically important shears and bending moments are easily calculated.

Finite Difference Approximation

The governing equation is $\dfrac{EId^4y}{dx^4} + ky - W(x) = 0$, and this must be expressed in finite difference form

0	h	1	h	2	h	3	h	4

Considering 5 adjacent points of subdivision, the finite difference form of $\dfrac{h^2d^2y}{dx^2}$ at the point 2 is $h^2\left(\dfrac{d^2y}{dx^2}\right)_2 = -2y_2 + y_1 + y_3.$

Differentiating twice and multiplying by h^2

$$h^4\left(\frac{d^4y}{dx^4}\right)_2 = -\frac{2h^2d^2y_2}{dx^2} + \frac{h^2d^2y_1}{dx^2} + \frac{h^2d^2y_3}{dx^2}$$

$$= -2(-2y_2+y_1+y_3)+y_2+y_0-2y_1+y_2+y_4-2y_3$$

$$= 6y_2-4y_1-4y_3+y_0+y_4.$$

$\therefore$ The equation expressed in finite difference form is

$$-6y_2+4y_1+4y_3-y_0-y_4+\left(\frac{Wh^4}{EI}\right)_2 - \frac{kh^4y_2}{EI} = 0 = F_2$$

The relaxation pattern for liquidating residuals will be for the typical point 2.

		$+1$		
-1	4	$-6-\dfrac{kh^4}{EI}$	4	-1
0	1	2	3	4

In order to interpret the boundary conditions we need also $\dfrac{d^2y}{dx^2}$ and $\dfrac{d^3y}{dx^3}$ expressed in finite difference form, as follows :—

$$h^2\left(\frac{d^2y}{dx^2}\right)_2 = -2\tfrac{1}{2}y_2+1\tfrac{1}{3}(y_1+y_3) - \frac{1}{12}(y_0+y_4)$$

$$h^3\left(\frac{d^3y}{dx^3}\right)_2 = 2y_0-4y_1+4y_3-2y_4.$$

[1] "Relaxation Methods in Engineering Science."—Southwell.

Boundary Conditions

The fomulæ defining the residuals and the relaxation patterns must be modified at the boundaries to conform with the boundary conditions, which are as follows :—

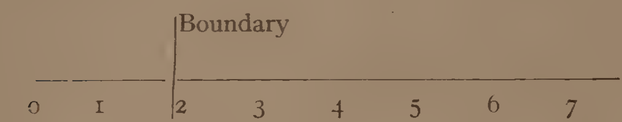
At the ends of the beam we have—

1. Zero Bending Moment, i.e., $$\frac{EId^2y}{dx^2} = 0$$

2. Zero Shear Force, i.e., $$\frac{EId^3y}{dx^3} = 0$$

3. $$\frac{d^4y}{dx^4} + \tfrac{1}{2}\,ky - \tfrac{1}{2}W\,(x) = 0$$

0 and 1 fictitious points outside the range



Expressing these in finite difference form with origin at point 2, i.e., on the boundary, we have :—

from 1) $(y_0+y_4) - 16(y_1+y_3) + 30y_2 = 0$
from 2) $2y_0 - 4y_1 + 4y_3 - 2y_4 = 0$
from 3) $-(y_0+y_4) + 4(y_1+y_3) - (6+k')\;y_2 = 0$

where $k' = \tfrac{1}{2}\left(\frac{kh^4}{EI}\right)_2$. It is assumed that there is no applied loading at the ends ; therefore there is no $W(x)$ term.

To eliminate y_0 and y_1 (fictitious points) we have—

4) $$y_0 = 4y_3 - 3y_4 - \frac{30k'}{12}(2y_3 - y_4)$$

5) $$y_1 = 3y_3 - 2y_4 - \frac{15k'}{12}(2y_3 - y_4)$$

6) $$y_2 = 2y_3 - y_4 - \frac{7k'}{12}(2y_3 - y_4)$$

The Residual at the point 3 is then

$$F_3 = -\left[1 + \left(\frac{kh^4}{EI}\right)_3\right] y_3 + 2y_4 - y_5 - \frac{13}{12}\,k'\;(2y_3 - y_4)$$

and $$F_4 = -\left[5 + \left(\frac{kh^4}{EI}\right)_4\right] y_4 + 2y_3 + 4y_5 - y_6 + \frac{7}{12}\,k'\,(2y_3 - y_4)$$

It is not necessary to record residuals on the boundary since the boundary deflection y_2 can be calculated from equation (6) when y_3 and y_4 are known.

The relaxation pattern for point 3 will be :—

2	3	4	5
Not recorded	$-\left[1+\left(\frac{kh^4}{EI}\right)_3+\frac{13}{6}k'\right]$	$\left[2+\frac{7}{6}k'\right]$	-1
	+1 here.		

And for point 4 :—

2	3	4	5	6
Not recorded	$\left[2+\frac{13}{12}k'\right]$	$\left[-5-\left(\frac{kh^4}{EI}\right)_4\frac{7}{12}k'\right]$	4	-1
		+1 here.		

If the simplifying assumption that $k' = 0$ is made, the above patterns are much less cumbersome. The loss of accuracy implicit in this assumption could probably be accepted in many cases arising in practice.

Statical Check

The total downward load must equal the total upthrust on the beam. This will not usually be the case on first calculation. If the discrepancy is small, as it should be, a correction can be made by adding the same deflection to each point. This will not sensibly affect the residuals.

Point Load

The loading intensity due to a point load W is taken as $\left(\frac{W}{h}\right)$ i.e., the equivalent of a distributed load of total value W spread over a length h.

Solution

$$\frac{kh^4}{EI} = \text{const.} = 0.01 \quad h = 1/12\;L.$$

Unit breadth.

857 1449½ 2042 2614½ 3129¼ 3522⅝ 3697¾
843 2651 3672
℄

3522⅝ 3129¼ 2616½ 2042 1449½ 857
2651 843

The figures given are the values of $N \times 52 \times 10^4$ in

$$y = \frac{NWL^3}{EI}$$ at each point of subdivision. The values underlined were obtained from the solution by pure mathematics. The maximum discrepancy, which occurs on the boundary, is 1.65 per cent.

The estimated Maximum Bending moment agrees with the true Bending Moment within one per cent.

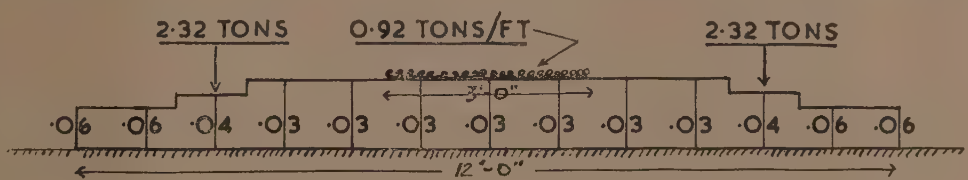
Unliquidated Residuals

The largest remaining residual could have been liquidated completely by an assumed variation of $1\frac{1}{2}$ per cent. in the value of k or EI.

A solution was obtained assuming $k' = 0$. This showed an error of $8\frac{1}{2}$ per cent. at the boundary, and about 2 per cent. at mid-span. The error in the Bending Moment in this case was one per cent.

The next problem considered is the more difficult one of a concrete beam of unit breadth whose Moment of Inertia varies along its length, loaded with a distributed load and two point loads.

In order to demonstrate the application of the Relaxation Method to such a problem, a hypothetical numerical example has been calculated. The data assumed for this purpose was as follows :



The value of the subgrade modulus k is taken as (200×12^3) lb./ft.^{3}.

The value of $\frac{kh^4}{EI}$ at mid-span is taken as 0.03.

The corresponding values of $\frac{kh^4}{EI}$ are shown at each point of subdivision on the diagram $(h = \frac{L}{12})$.

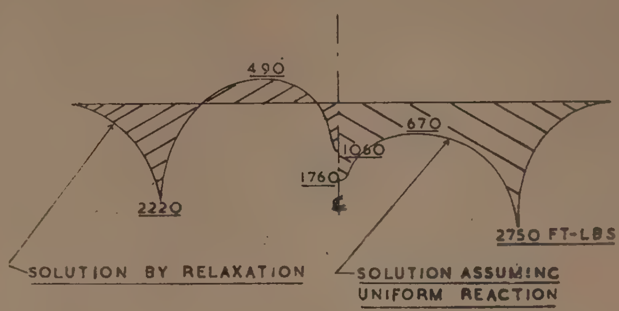
The boundary conditions are the same as in the previous problem with $k' = 0.03$.

An acceptable solution to the problem is

			℄			
3970	$5654\frac{1}{4}$	6280	$6566\frac{1}{4}$	6280	$5654\frac{1}{4}$	3970
$4848\frac{1}{2}$	$6036\frac{3}{4}$	$6487\frac{1}{8}$		$6487\frac{1}{8}$	$6036\frac{3}{4}$	$4848\frac{1}{2}$

The figures given are the value of y (ft.) multiplied by $(12^3 \times 10^4)$ at each point of sub-division.

Residuals. The largest unliquidated residual could have been completely removed by an assumed variation of one per cent. in the value of EI or k.



Bending moment diagram

It is interesting to note that in the example chosen the common assumption of a uniform base reaction fails to indicate the reversal in the sign of the bending moment which in fact occurs. This could have serious consequences if the foundation beam were of reinforced concrete.

Conclusions

The method seems well suited to solve the problem in question and represents a compromise between the usual crude design assumption of uniform base pressure and the solution by pure mathematics which entails a great deal of work for an accuracy which would be unnecessary in view of the uncertainty of the other factors.

The method is particularly apt where there is a possibility of reversal in the sign of the bending moment.

There are two special cases worthy of mention :

(1) $\frac{k}{EI}$ relatively large (2) $\frac{k}{EI}$ relatively small.

In case 1, it is possible that the ends of the beam will rise clear of the soil, i.e., negative deflections will be obtained. If this happens, the solution must be restricted to a length AB in the beam, since the soil cannot exert the downward force necessary to keep the beam in contact with it. Points A and B could be found by a trial and error process.

In case 2, the modification to the central operator (—6) in the finite difference equation will be small and it will be necessary to use sufficient significant figures in the solution to take account of this.

The Theory of Girder Walls with Special Reference to Reinforced Concrete Design

By H. L. B. Uhlmann, M.A.(Oxon.), Ph.D., A.M.I.Struct.E.

Synopsis

The ordinary theory of flexure in beams, based on the "straight line" theory of strain and stress distribution, is a particular case of the general theory, when the depth to span ratio is small. Design problems frequently arise, however, when this ratio is comparable to or greater than unity. To apply the "straight line" theory of flexure to such cases will lead to erroneous and often unsafe results. This paper sets out the main features of the design methods for beams in which the depth to span ratio is not small—henceforth to be designated "Girder Walls"—based in part on work already published [1, 2, 3], and in part on research work done by the writer.

The published work deals mainly with the theory of girder walls continuous over a large number of supports. The writer's work is concerned with the simply supported girder wall, the exterior span of a continuous girder wall, and the state of stress near a rectangular opening in such a member.

A brief indication of the method of solution of the stress fields is given. It will be noted that the principles employed are generally applicable to two-dimensional stress distribution problems, and are particularly useful when the boundaries of the members are irregular. Sufficient references are given to enable the reader to follow up the mathematical angle.

Scope and Applications

The walls of bunkers act as vertical girders spanning between column supports, and carry a portion of the floor load, a portion of the contained material and their own weight. They may be designed as girder walls without the necessity of beams or ribs except as archi tectural features. The theory is also applicable t curved walls where the radius of curvature is large com pared to the thickness of the wall, such as silos an hemispherical domes supported at isolated points alon their circumference.

In building construction it is often desired to have th lower floors entirely free of columns (e.g. in departmen stores, hotels, buildings housing a theatre, municipa buildings, etc.). Instead of heavy frame construction the use of Vierendeel Trusses in concrete or even struct ural steel trusses, it may be simpler to utilise the externa and partition walls as girders to span across the column free space and carry the rest of the building above them As rectangular openings for doors, windows, passages ducts, etc., will be required in such walls, special con sideration must be given to the design of reinforcemen near such openings.

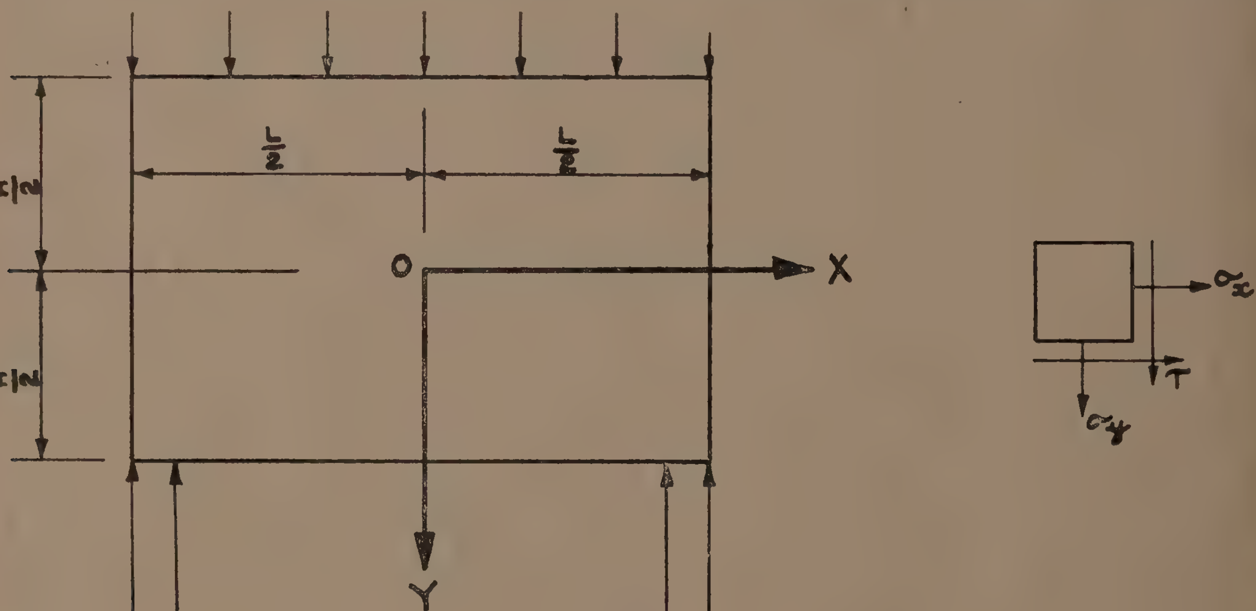


Fig 1.

These are typical examples of the application of th theory of girder walls. Yet, once one has acquire facility in the use of this theory, it is surprising how ofte the occasion for its use arises, and how often considerabl simplification of formwork, design and construction ma be effected, particularly in industrial structures.

Method of Solution

A simply supported girder wall under a typical loadin is shown in *Fig.* 1. The axes of reference, the dimension and the stress notation are as indicated. The thicknes of the "plate" is small compared with the length L an the height H. The problem therefore reduces to one o plane stress in which the variation of the stress com ponents σ_{xx}, σ_{yy} and τ_{xy}, across the thickness may b neglected and $\sigma_{zz} = \sigma_{yz} = \sigma_{zx} = 0$. Such a two-

[1] Reference numbers such as these refer to the Bibliography.

nensional system may be expressed in terms of the ry stress function F such that

$$\sigma_{xx} = \sigma_x = \frac{\partial^2 F}{\partial y^2} \quad \ldots \ldots \ldots \ldots \quad (1)$$

$$\sigma_{yy} = \sigma_y = \frac{\partial^2 F}{\partial x^2} \quad \ldots \ldots \ldots \ldots \quad (2)$$

and $$\tau_{xy} = \tau = \frac{\partial^2 F}{\partial x \partial y} \quad \ldots \ldots \ldots \ldots \quad (3)$$

hen no " body " forces are acting.

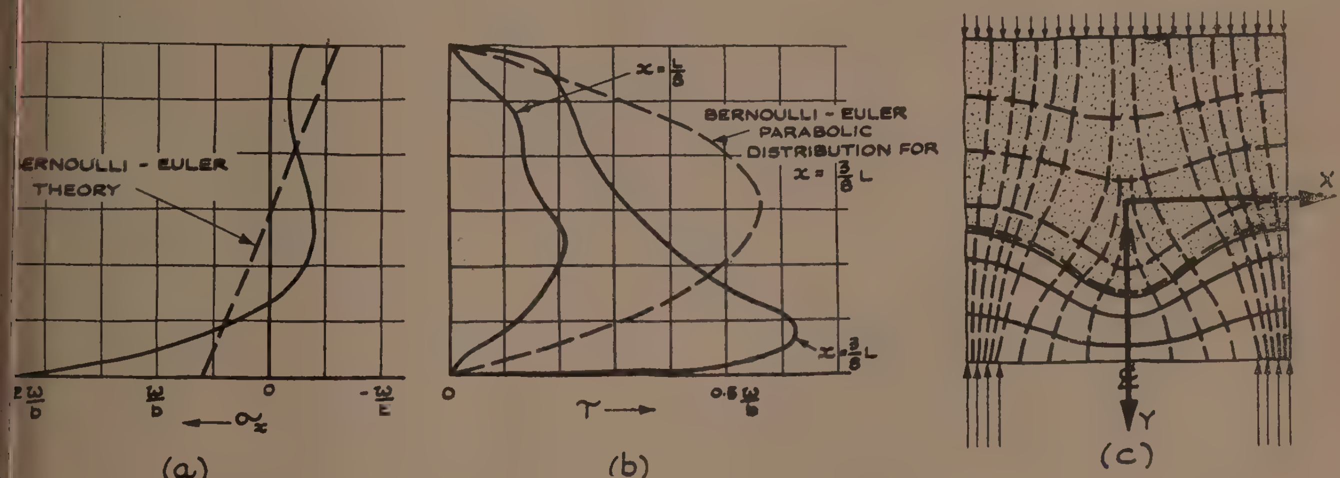
For equilibrium of the stress components and com- atibility of the deformations, F must satisfy the quation

$$\frac{\partial^4 F}{\partial x^4} + 2.\frac{\partial^4 F}{\partial x^2 \partial y^2} + \frac{\partial^4 F}{\partial y^4} = \nabla_1{}^4 F = 0 \quad \ldots \ldots \quad (4)$$

s well as the boundary conditions.

the lattice and the known values at the boundary points are inserted. Then by various definite procedures the approximate values in the table are modified, causing them to approach without limit to the true finite difference integral. A number of " dodges " are used to speed up convergence, such as removal of the first and second modes of the error function, and the use of a coarse and a fine lattice. For a full discussion of the theory involved the reader is referred to Richardson's paper.

This method is successful as long as the boundary stresses are determinate. When there are more than two supports solution is obtained by the application of Castigliano's minimum strain energy theorem together with the superposition of several stress fields each solved by Richardson's method. Details of the technique of approximation will not be given. Suffice to record in C. E. Inglis' words that " arithmetic solution comes not forth save by much prayer and fasting."



Bending Stresses σ_x across $\frac{C}{L}$

Shear Stresses τ at $x = \frac{3}{8}L$ and $x = \frac{L}{8}$

Stress Trajectories
Tensile Principle Stresses - Full
Compressive ,, ,, - Dotted
Compressive Field - Spotted

Fig 2. Uniformly distributed load on upper edge, $\frac{H}{L} = 1$

An investigation showed that the usual series solution, composed of products of circular and hyperbolic functions is able to satisfy the boundary conditions (the normal and shear stresses) at the boundaries $y = \pm \frac{H}{2}$ only. No constants are then left to prescribe the stresses at the boundaries $x = \pm \frac{L}{2}$ and the solution is applicable to continuous girder walls only. This is the reason why no solution for a simply supported wall girder has as yet been put forward.

Various methods of solution were tried, among them Timoshenko's minimum strain energy method [4], and Southwell's " Relaxation of Constraints "[5]. These were found to be more laborious and less successful than Richardson's method of successive approximation[6], which was finally adopted. The outline of the process is as follows :—

As in any finite difference method of handling differential equations, the continuum of points on the boundaries and in the interior of the region is replaced by a discrete set of points. A rectangular network is laid down over the region. Approximate (guessed) values are assigned to the Airy function at the interior nodes of

Results of the Stress Analysis

Figs. 2-5 have been selected to illustrate typical stress configurations. The height to length ratio $\frac{H}{L}$ is unity throughout.

Fig. 2 shows stress distributions for uniformly distributed load on the upper edge. The lines corresponding to the ordinary Bernoulli-Euler theory for homogeneous materials are superposed for comparison. Note the lowering of the " neutral axis " in (*a*) and the shear stress concentration near the lower edge in (*b*). The latter occurs only near the support. Near the centre of the span the distribution is more nearly parabolic. This shear stress concentration occurs near the point of application of any concentrated load.

Fig. 3 illustrates the change in the stress trajectories when the loading is applied at the lower edge. As is to be expected the field is much more tensile in character.

Fig. 4 indicates how radically the stress distribution varies for different types of loading although the $\frac{H}{L}$ ratio is unchanged. The bending stresses across the centre line form two tension and two compression zones.

A more comprehensive picture of the stress distribution is shown by the trajectories. It will be seen that the lower horizontal compression zone forms an "island" with "horizontal" tension occurring not far from the centre line.

Fig. 5 applies to a continuous girder wall on three supports at the same level. Note that the support section I—I has two compression zones and a high tension zone not far from the lower edge. The main tensile reinforcement should therefore be spread over a region starting immediately above the neutral axis, and not along the upper face as for ordinary beams.

Sufficient illustrations have been given to show that the stress distribution in beams undergoes a fundamental change when the height to span ratio is no longer small. The simple straight line theory, and with it the ordinary formulæ used for detailing the reinforcement in beams, are no longer applicable. By using the familiar concept of the "lever arm" and some simple geometrical constructions, and by making some justifiable assumptions, a fairly straight forward design method can be evolved. This is given in the next section in the form of

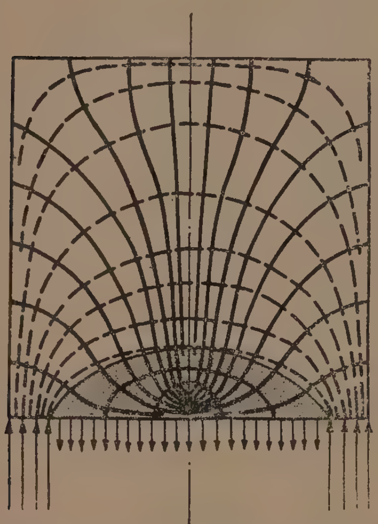
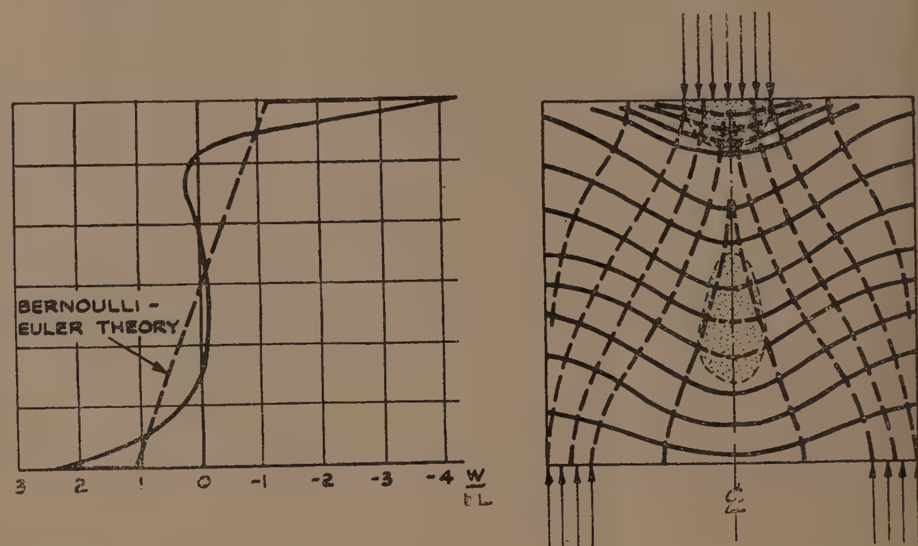


Fig. 3. Stress trajectories: uniformly distributed load on lower edge $\frac{H}{L} = 1$

Tensile field shaded



(a)

Bending stresses σ_x across $\frac{C}{L}$

(b)

Stress trajectories

Fig. 4.—Concentrated load on centre of upper edge, $\frac{H}{L} = 1$

Recommendations for Design

The following rules and accompanying diagrams contain information for the design of reinforced concrete girder walls with some notes on the detailing of reinforcement near rectangular holes. The data is partly based on an exploration of the stress fields, found by the methods indicated in the last two sections, and partly abstracted from Dischinger's papers [2, 3].

The principles underlying the design of the steel reinforcement are as follows: The stress distribution is determined on the assumption that the elastic theory for a homogeneous material is applicable. Concrete is assumed to be incapable of taking tension, and all tension regions as found by the elastic theory are reinforced with steel on the basis that full working stress is developed. This method of calculating reinforcement appears to be generally accepted wherever the ordinary concrete flexure theory is not applicable.

Clause 1 applies to girder walls of uniform section only; Clauses 2 to 5 apply to girder walls (*a*) of uniform section; (*b*) of uniform section in the direction of the span but varying in thickness with height.

CLAUSE 1. The Minimum Thickness as determined by Elastic Stability.*

Where the minimum wall thickness is determined by transverse bending stresses (as in bunkers, tanks, etc.), the possibility of buckling need not be considered.

Where no major transverse bending stresses exist, the elastic stability should be checked. The following formula holds for a panel loaded in its own plane and simply supported laterally on all four sides. Such lateral supports may be cross-walls and floors. The existence of adjacent continuous panels is immaterial.

The minimum permissible thickness b' is given by

$$b' \geqslant 0.06 \frac{L}{\sqrt{K}} \quad \ldots \quad (5)$$

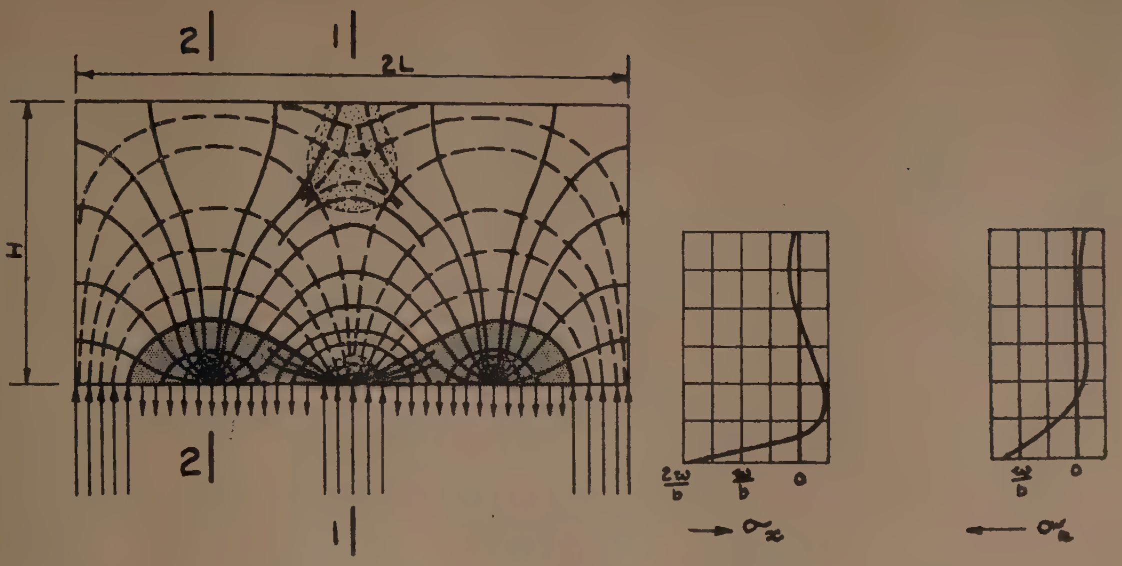
where L = length of panel
H = height of panel

and K is a function of $\frac{H}{L}$ given in following table:

*Based on references 7 and 8.

TABLE 1

$\frac{H}{L}$ =	0.2	0.3	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0	2.2	2.4	2.7	∞
K =	22.2	10.9	6.92	4.23	3.45	3.29	3.40	3.68	3.45	3.32	3.29	3.40	3.32	3.29	3.29



(a)
Stress Trajectories

(b)
Bending Stresses
σ_x on 1–1

(c)
Bending Stresses
σ_x on 2–2

Fig. 5.—Continuous girder wall on 3 supports at the same level under uniformly distributed load on bottom edge, $\frac{H}{L}$ = 1

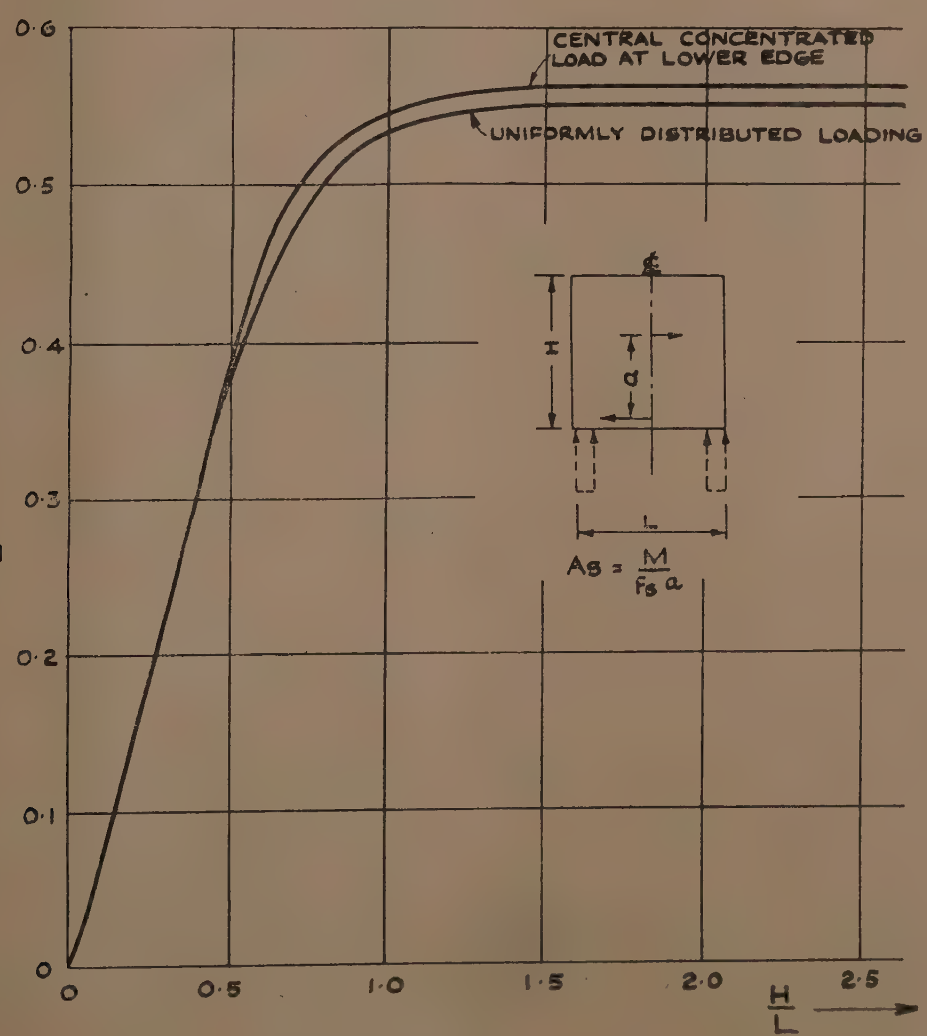


Fig. 6.—Lever arm curves for (1) Uniformly distributed load (2) Central concentrated load at lower edge

This result is independent of the loading as it is assumed that compressive working stresses are reached in the girder wall.

CLAUSE 2. Simply Supported Girder Walls

A. Uniformly Distributed Loading along Upper Edge

(1) *Bending Reinforcement :*

The necessary area of tensile bending steel A_s is obtained from the relation

$$A_s = \frac{M}{f_s . a} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

where M is the bending moment at the centre section, f_s is the permissible steel stress and a is the lever arm as given in *Fig.* 6. The bars are to be placed along the lower edge and are to be bent up at 45 deg. strictly in accordance with the ordinary bending-up diagram found from the bending moments.

(2) *Vertical hanging and " inclined tension " *steel :*

None required.

B. Uniformly Distributed Loading along Lower Edge.

(1) *Bending reinforcement :*

As for A(1).

(2) *Vertical hanging and inclined tension steel :*

The area of hanging steel A_h is given by

$$A_h = \frac{W}{f_s} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

where W is the applied load between the supports and f_s is the permissible steel stress. The bars are to be vertical and uniformly spaced, and may be stopped off or bent outwards at 45 deg. along the lateral limitation lines AF, the construction for which is shown in *Fig.* 10.

The bent-up bending steel is insufficient to take care of the inclined tension. The additional reinforcement required for this purpose is read off from *Fig.* 8. These bars should cross the lateral limitation lines AF between K and F, and should be more closely spaced at K than at the upper edge. Small adjustments are permissible near K to secure continuity between the additional inclined tension steel and the bent-up bending steel. Hanging steel crossing the lateral limitation lines may be bent over at 45 deg. to help take care of inclined tension. The balance of the hanging steel may be stopped off in accordance with the vertical tension distribution curves of *Fig.* 11, bent over bars to be regarded as stopped off.

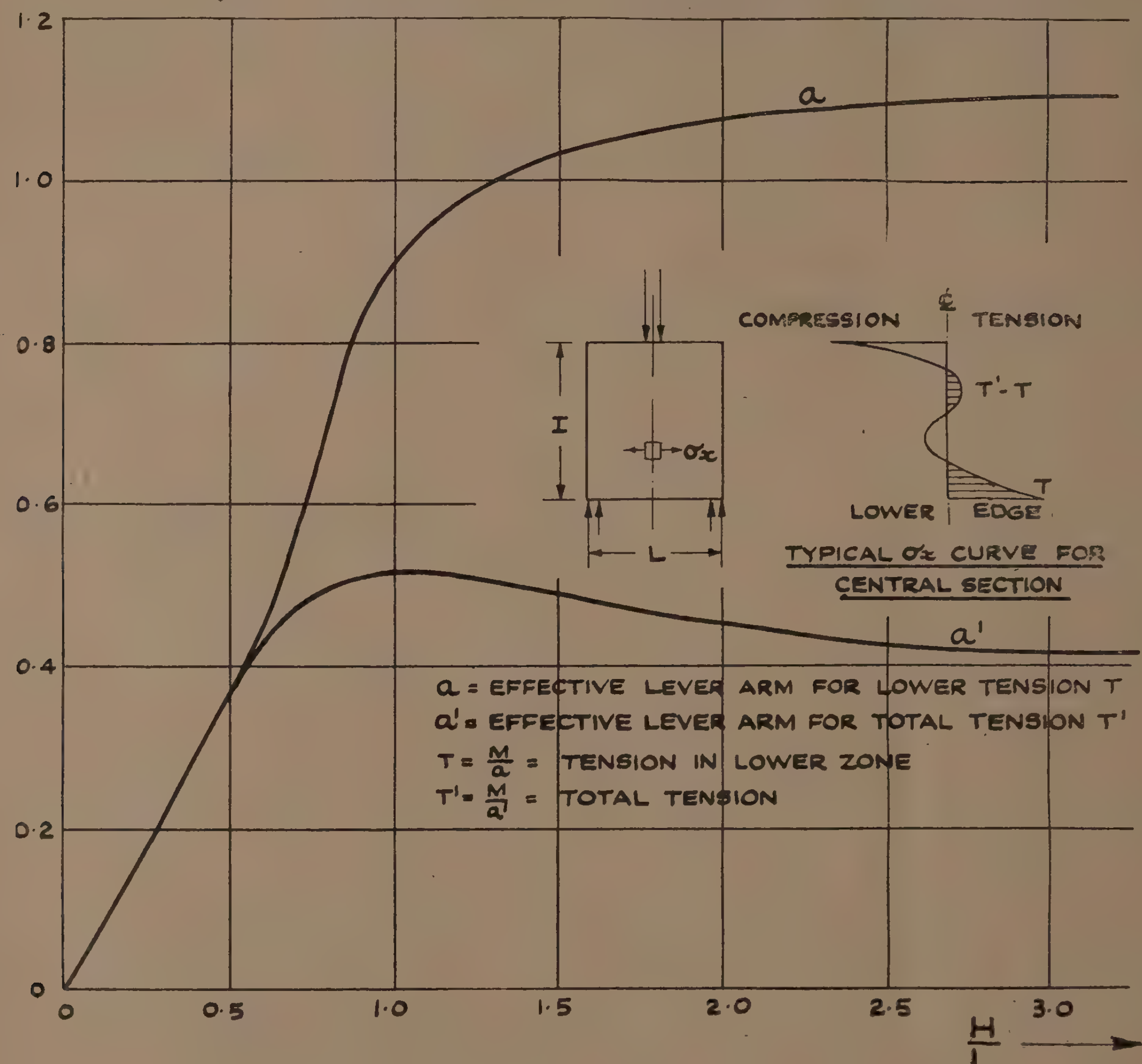


Fig. 7—Effective lever arm curves for upper central concentrated load

C. Concentrated Load at Centre of Lower Edge.

(1) *Bending reinforcement :*

Procedure as in A(1). The lever arm curve is shown in *Fig.* 6.

(2) *Vertical hanging and inclined tension steel :*

The area of hanging steel A_h is given by equation (7) where W is the applied concentrated load. The bars,

*Analogous to " diagonal tension."

concentrated in the region of the applied load, are to be carried up vertically and are to be bent outwards or stopped off in accordance with the vertical tension distribution diagram of *Fig.* 13.

The additional inclined bending steel is read off from *Fig.* 8. These bars should cross the lateral limitation lines AF between K and F (as shown in *Fig.* 12), and

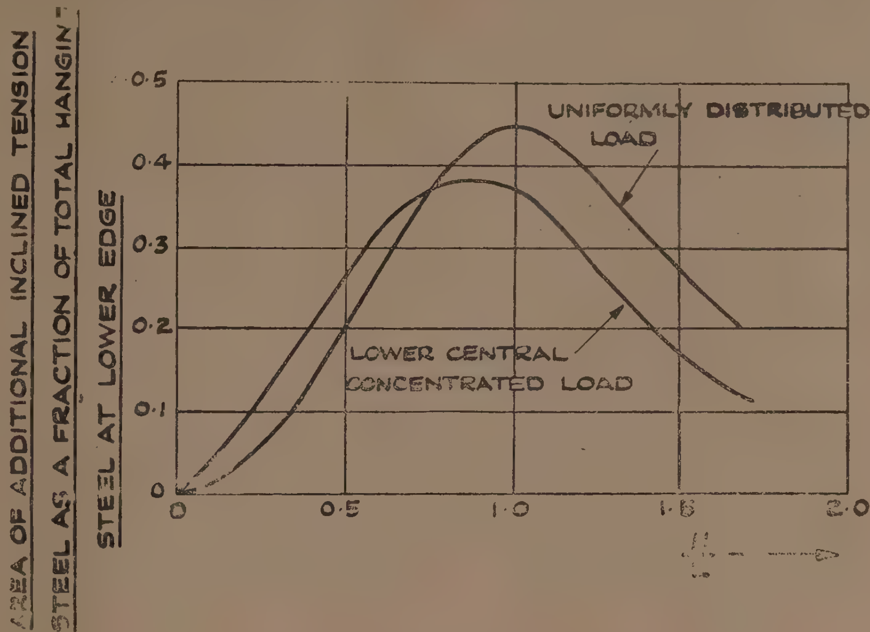


Fig 8. Additional inclined tension steel

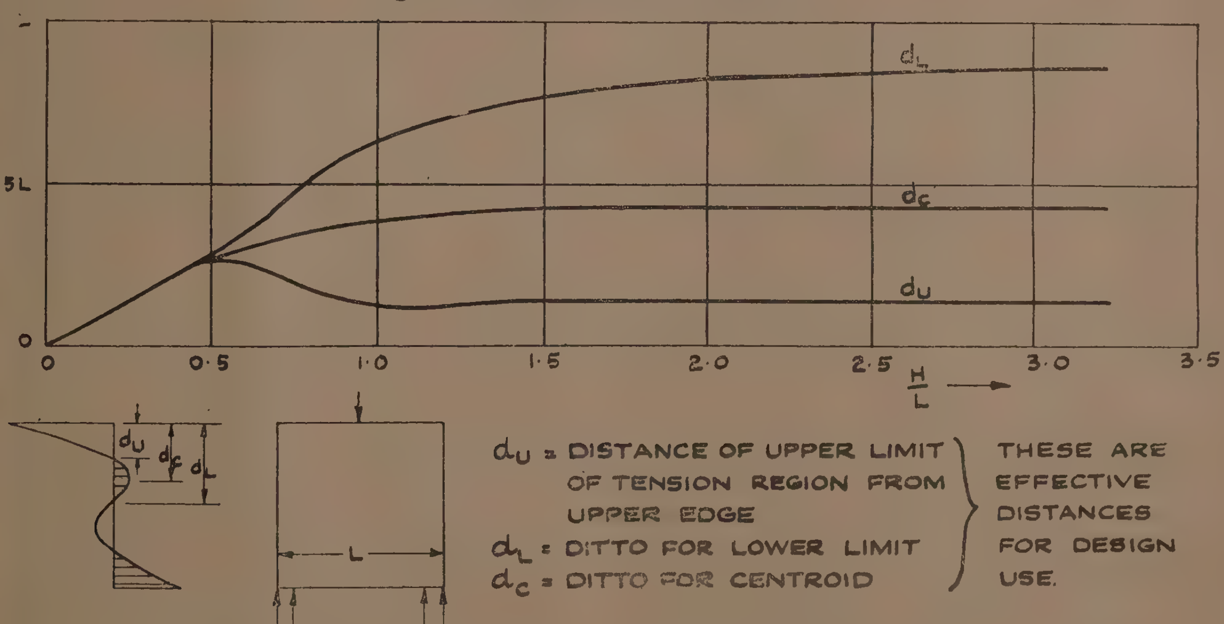


Fig 9. Effective limits and centroid of upper tension zone

should be more closely spaced at K than at the upper edge. Small adjustments are permissible near K to secure continuity between the inclined tension steel and the bent-up bending steel.

Hanging steel crossing the lateral limitation lines may be bent over at 45 deg. to help the care of inclined tension.

D. Concentrated Load at Centre of Upper Edge.

(1) *Bending reinforcement* :

For $\dfrac{H}{L} > \frac{1}{2}$ the total bending tension T' may be considered as split into a lower tension zone T and an upper tension zone $(T'\text{-}T)$.

The lower area of tension steel A_L (corresponding to T) is calculated as before from the formula

$$A_L = \frac{M}{f_s . a} \quad \ldots\ldots\ldots \quad (8)$$

where M is the central bending moment, a is the equivalent lever arm shown in *Fig.* 7 and f_s is the permissible steel stress. This lower zone steel is to be bent up strictly in accordance with the ordinary bending-up diagram found from the bending moments.

The upper area of tension steel A_U (corresponding to $T'\text{-}T$) is calculated by first finding the total steel area A'_T from the formula

$$A'_T = \frac{M}{f_s . a'} \quad \ldots\ldots\ldots \quad (9)$$

where a' is the equivalent lever arm shown in *Fig.* 7.

Then $A_U = A_T' - A_L$ $\quad \ldots\ldots\ldots \quad$ (10)

The effective upper and lower limits of the upper tension zone, d_U and d_L, as well as the position of the centroid of tension d_C, all measured as distances below the upper edge, are plotted in *Fig.* 9. The reinforcement A_U is to be spread over this region with its centroid a distance d_C below the upper edge. The bars are to be bent up at 45 deg. as before, starting with the upper bar, in accordance with the bending-up diagram.

(2) *Vertical hanging and inclined tension steel :*
None required.

E. Combinations of Loadings Types A, B, C and D.

(1) *Bending Reinforcement :*
Separate out the loading into type A, B, C and D components. Calculate the steel areas required for each separately. Combine to obtain the steel for the lower and upper tension zones A_L and A_U respectively. The lower bending steel A_L is bent up according to the bending-up curve based on the combined bending moment diagram. The upper bending steel A_U is bent up according to the bending-up curve based on the upper concentrated central load only.

(2) *Vertical hanging and inclined tension steel :*

Separate out into the component loadings as in (1) and detail separately according to the rules given for loadings types A, B, C and D. The systems of reinforcement are then superimposed. Adjustments and rationalisation may then be necessary to secure continuity of reinforcement.

For detailed information on vertical tension distribution diagrams for compound loading the reader is referred to a more comprehensive series of articles and diagrams on Girder Walls by the writer, kept for reference in the Institution Library.

CLAUSE 3. Girder Walls Continuous over a large number of supports and loaded by (a) A Uniformly Distributed Load, (b) Concentrated Loads at the centre of every Span applied at the Lower Edge.*

(1) *Bending Reinforcement :*

A reference to *Fig.* 14 (b) indicates that in order to detail the steel at the centre line of the span the bending moment and the lever arm only are required. All reinforcement is to be placed close to the lower edge. On the other hand it is clear from *Fig.* 14 (c) that additional information is required concerning the location of the reinforcement. For this purpose the height of the "neutral axis" y_0 and the centre of tension y_1 above the lower edge will be given. It will be noted that the dimensions tabulated are a function of

$$\varepsilon = \frac{c}{L} = \frac{\text{support width}}{\text{span}}.$$ The value $\varepsilon = \frac{1}{2}$ will be found of great value when live loads are considered.

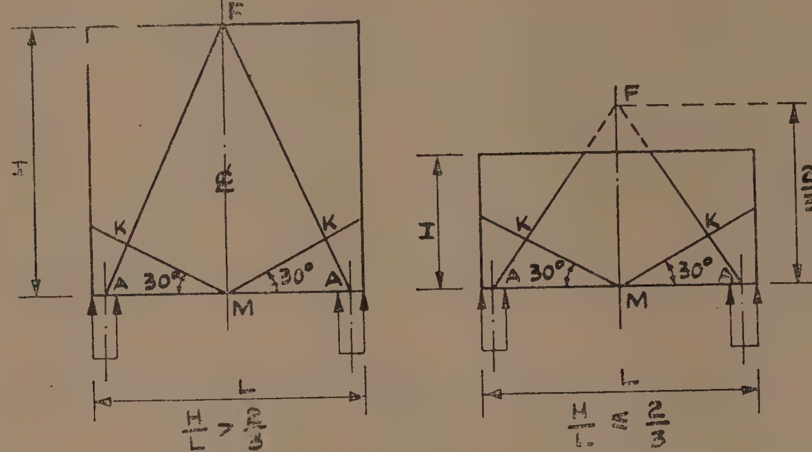
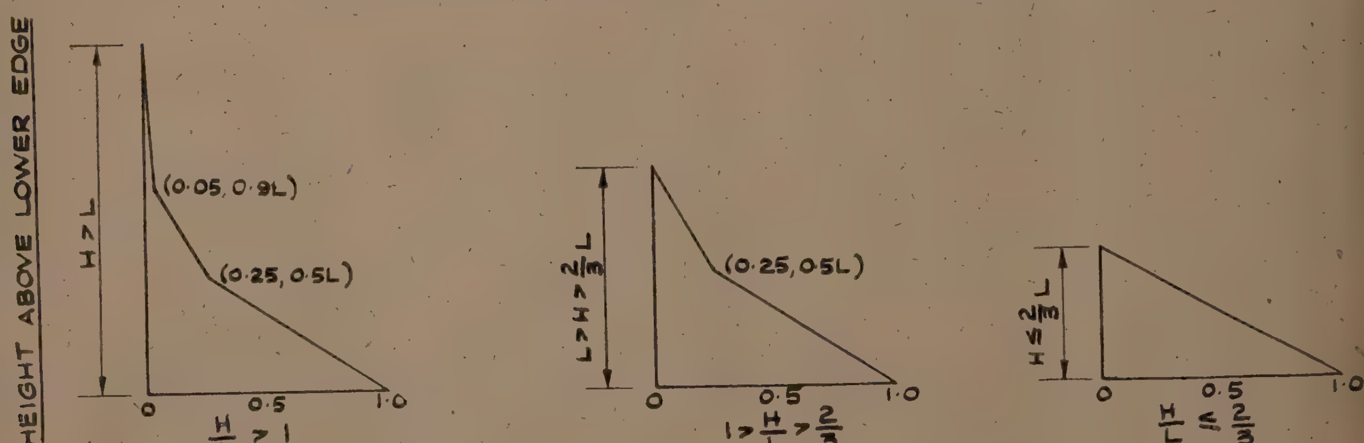


Fig 10. Lateral limitation lines AF for uniformly distributed load



Fraction of support reaction or tension across lower edge of girder wall

Fig. 11.—Vertical tension and compression distribution diagrams—uniformly distributed load

*This clause is based on Dischinger's papers (References 2 and 3).

The rule is as follows :—
The lever arm a, the height of the "neutral axis" y_0 and the height y_1 of the centroid of the tension region at the support section above the lower edge are given by the Bernoulli-Eulerian values

$$a = \frac{2}{3} H$$
$$y_0 = \frac{1}{2} H \qquad (11)$$
$$y_1 = \frac{5}{6} H$$

or by the values given in Table 2, whichever is smaller.

(1) *Support reactions :*

The end support is to be designed for a pressure of $\frac{5}{8} W$, the first and all other interior supports for a pressure of W, where W is the total load per span.

(2) *Bending, vertical hanging and inclined tension reinforcement :*

The outer half of the end span is to be designed as if the whole span were simply supported. This consideration determines the maximum (central) bending moment. The lever arm and the method of bending up etc. are as for Clause 2.

TABLE 2

Type of Loading	Section	Bending moment (sagging + ve)	$\varepsilon = \frac{c}{L} =$	$\frac{1}{2}$	$\frac{1}{5}$	$\frac{1}{10}$	$\frac{1}{20}$	$\frac{1}{\infty}$
Uniformly distributed load w per unit length (See Fig. 14).	span	$+ \frac{w L^2}{24} (1 - \varepsilon^2)$	$\frac{a}{L}$	0.437	0.465	0.468	0.469	0.470
	support	$- \frac{w L^2}{24} (1-\varepsilon)(2-\varepsilon)$	$\frac{a}{L}$	0.437	0.373	0.337	0.312	0.302
			$\frac{y_0}{L}$	0.163	0.100	0.065	0.048	0.038
			$\frac{y_1}{L}$	0.491	0.403	0.356	0.318	0.280
Central concentrated load W applied at lower edge† (width of applied load = c).	span	$+ \frac{W L}{8} (1-\varepsilon)$	$\frac{a}{L}$	0.437	0.420	0.412	0.405	0.395
	support	$- \frac{W L}{8} (1-\varepsilon)$	$\frac{a}{L}$	0.437	0.420	0.412	0.405	0.395
			$\frac{y_0}{L}$	0.163	0.135	0.100	0.088	0.072
			$\frac{y_1}{L}$	0.491	0.456	0.435	0.418	0.400

†For central concentrated loads applied at the upper edge the same results as for central concentrated loads applied at the lower edge may be used, provided $\frac{H}{L} < 1$. For $\frac{H}{L} > 1$ no definite information is available. Recourse may be had to an analogy with the results given in Clause 2D for simply supported girder walls, both for span and support sections.

Fig. 15 indicates diagrammatically the location of the reinforcement for a continuous girder wall under a uniformly distributed loading along the upper edge. Account should be taken of the spread of the tensile forces above the supports by making use of radial lines such as k.

CLAUSE 4. The End Span of a Continuous Girder Wall on three or more Supports.

Girder walls are very sensitive to settlement. Where support reactions vary, as in end spans, the mere elastic differential deflection of the supporting columns suffices to alter the stress distribution radically. In end spans allowance must therefore be made for "elastic settlement."

The inner half of the exterior span, including the first support section, is to be designed as an interior span of a continuous girder wall (Clause 3). The reversed bending moment at the first interior support section may be taken the same as for long beams.

To provide against settlement of the first interior support the end span and the first interior span together are to be regarded as a simply supported girder wall of length 2L, loaded by a central concentrated load $\frac{1}{4} W$.

The corresponding area of bending steel, etc. are to be found as in Clause 2 C. When the support foundations

are "unyielding" this settlement allowance may be halved.

CLAUSE 5. Design near an opening in a Girder Wall.

Referring to *Fig.* 16 let o1 and o2 be the average directions of the maximum and minimum principal

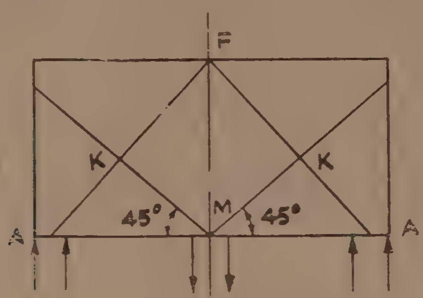


Fig 12. Lateral limitation lines AF for all $\frac{H}{L}$ ratios for lower central concentrated load

stresses respectively in the region of the opening when the member is regarded as unperforated. Then the "unperforated force"* intercepted by the hole in each direction will be called the "Intercepted Force."

*The area integral of the stresses in the same region of the member before perforation.

The effect of an opening on the unperforated stress distribution is as follows :—

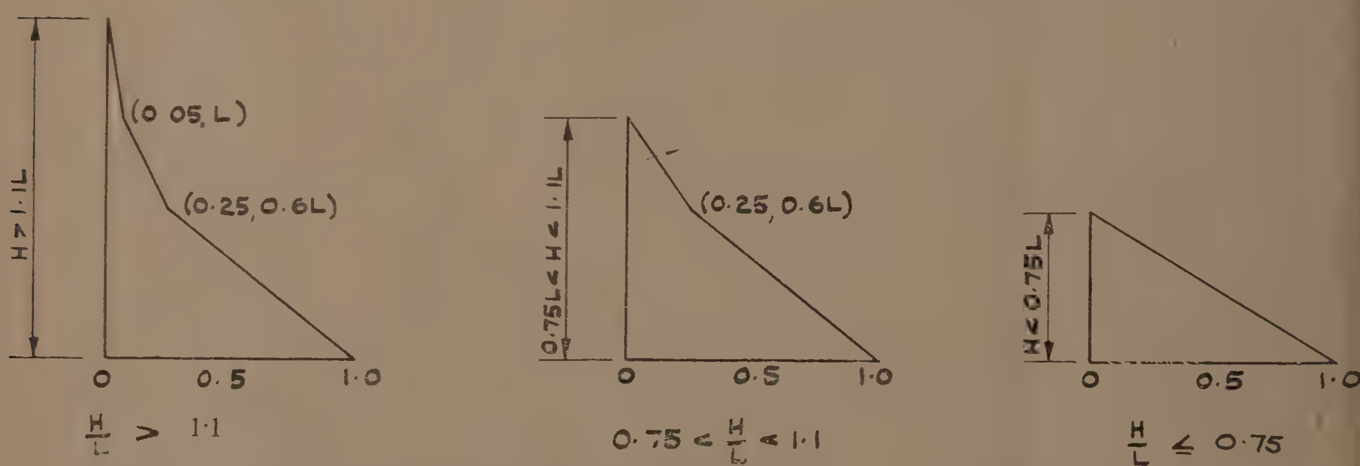
(1) The Intercepted Force is deviated past the opening on each side, leading to a rise of stress along those edges of the hole which are approximately tangential to the unperforated lines of stress (near corners A and A' in *Fig.* 16).

(2) A force of opposite sign is induced along the edge of the opening approximately perpendicular to the unperforated lines of stress considered (near corners B and B').

Let the Intersepted Tension along o1 be P_1, the Intercepted Comprescion along o2 be P_2. Consider first the effect of the hole on the tensile stresses o1 only.

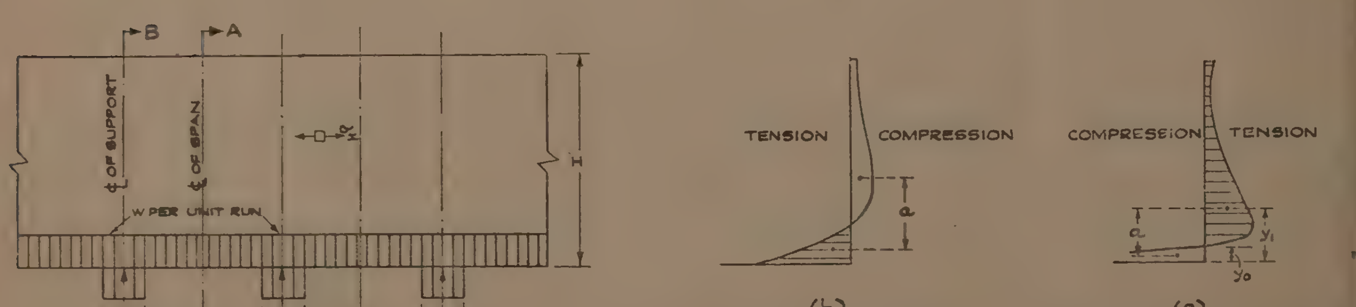
The tensions *additional* to those existing before perforation, which are deviated past the hole at A and A' will be expressed by means of the products $+\Delta_A.P_1$ and $+\Delta_A'.P_1$ (tension is taken positive), where Δ_A and Δ_A' are the "Deviation Factors" at A and A' respectively. The force of opposite sign (a compression in this case) that is "induced" at B and at B' will be expressed by means of the product $-\delta_1 . P_1$, where δ_1 is the Induction Factor corresponding to the intercepted force P_1.

Similarly the *additional* forces produced by the hole on the compressive stresses o2 are $-\Delta_B . P_2$, $-\Delta_B' . P_2$ and $+\delta_2 . P_2$ at B, B' and A and A' respectively.



Fraction of support reaction or tension across lower edge of girder wall

Fig. 13.—Vertical tension and compression distributions diagrams for lower central concentrated load



(a)

UNIFORMLY DISTRIBUTED LOADING G. W. CONTINUOUS OVER MANY SUPPORTS $\epsilon = \epsilon$.

(b)

BENDING STRESS σ_x ACROSS MIDSPAN SECTION A-A, G.W. CONTINUOUS OVER MANY SUPPORTS. $\frac{H}{L} = 1$, $\epsilon = \frac{1}{10}$ UNIFORMLY DISTRIBUTED LOAD

(c)

BENDING STRESS σ_x ACROSS SUPPORT SECTION B-B, G.W. CONTINUOUS OVER MANY SUPPORTS. $\frac{H}{L} = 1$, $\epsilon = \frac{1}{10}$ UNIFORMLY DISTRIBUTED LOAD

Fig 14.

Hence the nett *additional* forces produced by the presence of the opening are :

$$\begin{array}{ll} \text{at } A: & +\Delta_A \cdot P_1 + \delta_2 \cdot P_2 \\ A': & +\Delta_A' \cdot P_1 + \delta_2 \cdot P_2 \\ B: & -\Delta_B \cdot P_2 - \delta_1 \cdot P_1 \\ B': & -\Delta_B' \cdot P_2 - \delta_1 \cdot P_1 \end{array} \qquad (12)$$

The definition of " Intercepted Force " given above must be modified when the " unperforated " stress in the direction considered passes through a zero value within the area delimited by the hole. When the Deviated Force is concerned the tensile portion of the force intercepted by the opening will be taken as the Intercepted Force. When, on the other hand, the Induced Force is concerned, the compressive part will be taken as the Intercepted Force.

1. Values of the Deviation Factor Δ

TABLE 3

Type of " unperforated " stress field	Values of Δ
Principal stresses decreasing as the edge of the hole is approached*	$0.5 < \Delta < 0.75$
Principal stresses increasing as the edge of the hole is approached	$\Delta < 0.5$
Principal stresses constant in the vicinity of the hole	$\Delta = 0.5$

*Stresses are considered across a section through the hole perpendicular to the Deviated Force under consideration.

The values given above are not sensibly affected by the proximity of an external boundary. The upper limit of the Deviation Factor is 0.75.

2. Values of the Induction Factor δ

This factor varies between zero and 0.15 depending on the shape of the hole. The value $\delta = 0.15$ is applicable

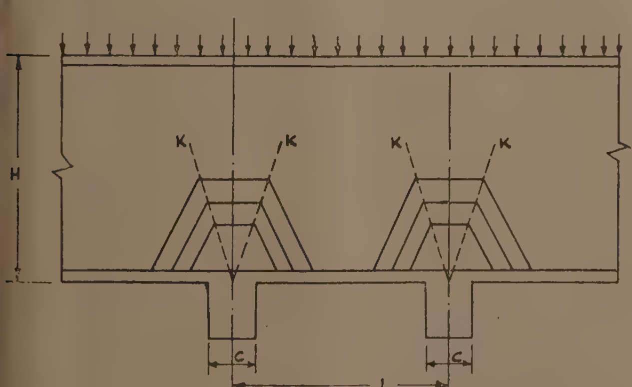


Fig 15 Reinforcement scheme for continuous girder wall under uniformly distributed loading along the upper edge

to a crack perpendicular to the lines of stress considered. As the factor is small the maximum value may be used with little loss of economy.

3. General

The reinforcement throughout is then based on the relation $A_s = \dfrac{\text{tensile force}}{\text{permissible steel stress}}$

For guidance in determining the unperforated stress fields in a girder wall the reader is referred to the writer's articles in the Institution Library.

Conclusion

Confirmation of the stress distributions on which the above design recommendations are based was obtained by running a series of tests on simply supported reinforced concrete girder wall models with and without openings. Substantial agreement with the theoretical results was obtained. For experimental details the reader is again referred to the articles in the Institution Library.

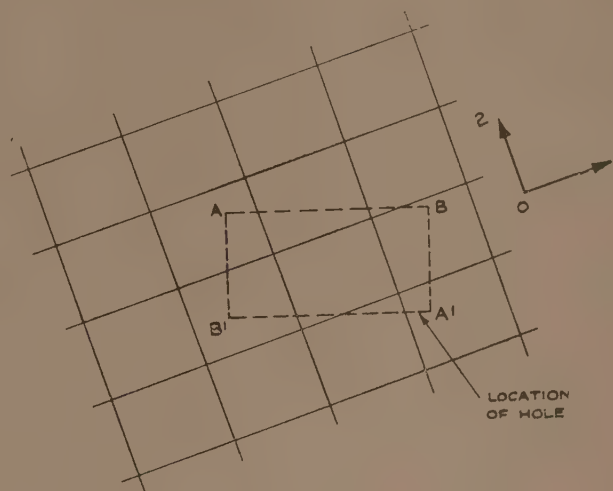


Fig 16. Showing location of opening in " unperforated " stress field

It is clear that the work on this subject is by no means complete. In view of the considerable labour involved in obtaining a solution for each load configuration, only the more immediately useful types of loading were considered.

It is hoped, however, that the data presented in this paper will cover most practical cases designers may encounter, and will assist in the advancement of this branch of engineering science.

Bibliography

[1] " Der gerade Stab. mit Rechteckquerschnitt als ebenes Problem," by F. Bleich, Bauingenieur, 1923, pp. 255-9, 304-7 and 327-31.

[2] " Beitrag zur Theorie der Halbscheibe und des wandartigen Trägers," by F. Dischinger, Publications of the International Association for Bridge and Structural Engineering, vol. 1, 1932, Zurich.

[3] "Die Ermittlung der Eiseneinlagen in wandartigen Trägern," by F. Dischinger, Beton und Eisen, 1933, pp. 237-239.

[4] " The Approximate Solution of Two-dimensional Problems in Elasticity," by S. Timoshenko, PHIL. MAG., vol, 47, 1924, pp. 1095-1104.

[5] " Relaxation Methods applied to Engineering Problems : III Problems involving Two Independent Variables," by D. G. Christopherson and R. V. Southwell, Proc. Roy. Soc., London, A vol. 168, 1938, pp. 317-350.

[6] " The Approximate Arithmetical Solution by Finite Differences of Physical Problems involving Differential Equations," by L. F. Richardson, Phil. Trans. Roy. Soc., London, A vol. 210, 1910.

[7] " The Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill, 1936.

[8] " Buckling of a Rectangular Slab subjected to Variable Marginal Loading," by Shizuo Ban, Publications, vol. 3, International Association for Bridge and Structural Engineering, 1935.

The Assumed Deflection Method for the Determination of Transverse Stresses in Slabs Supported on Two Sides*

By Ronald Noble, B.Sc.(Eng.)

The author wishes to make it clear that no more accuracy than is necessary for design purposes is claimed for this method.

He does claim, however, that use of it is simpler than the solution of a fourth order partial differential equation, and that all assumptions made in it are reasonable.

The only unknown quantity involved is the choice of point load, "P," and this must be left to the intelligent discretion of the designer.

General Description of Method

The essence of this method is contained in a consideration of the equilibrium of a beam, loaded at its centre with a point load "P," and supported elastically along its length in such a way that the degree of support is directly proportionate to the depression of the supporting medium.

It is assumed that the beam will bend into a segment of a circle under this loading, and it now remains to find the values of "l" and "$d\ max$" (chord length and segment height respectively) WHERE "l" IS NOT NECESSARILY THE SAME AS THE LENGTH OF THE BEAM. (Fig. 1.)

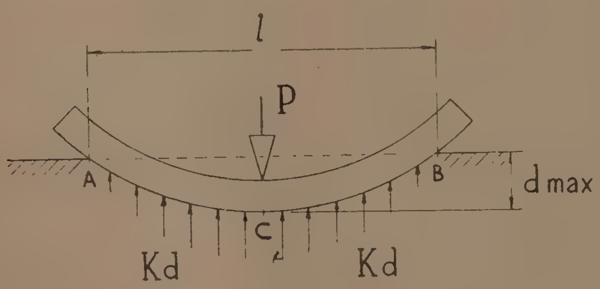


Fig. 1

Let the elastic constant of the support be "K". that is, the reaction between the beam and the support at any point is given by

$R = Kd$ where "d" is the deflection.

If we ignore the self weight of the beam as not affecting its deflected form, we may write down the first static equation, which is :—

$$\Sigma Kd = P \qquad (1)$$

We now consider the deflection due to this loading, and, since the arrangement is symmetrical, it is sufficient to consider only one half of the beam, and the problem now resolves itself into finding the deflection of the free end of a cantilever, length $\frac{l}{2}$, loaded with segmental loading of maximum value $Kd\ max$.

From this, the second static equation is obtained :—

$$(\text{deflection due to } Kd) = d\ max \qquad (2)$$

From equations (1) and (2), it is a simple matter to solve for "$d\ max$" and "l" and thence the value of the maximum Bending Moment in the beam, the stresses, etc., may be obtained.

Particular Application to the Slab Supported on Two Sides

Consider a slab of dimensions L_L and L_T supported along opposite sides of length L_T and of moderate skew. The dimensions L_L and L_T being measured parallel to the sides and to the supports of the slab respectively.

The principle assumption is that the slab approximates in action to a series of disconnected beams of unit width, parallel to the sides of the slab and of moment of inertia I_L, supported over a span L_L. with a series of disconnected unit beams of moment of inertia I_T and length L_T resting on them parallel to the supports. (Fig. 2.)

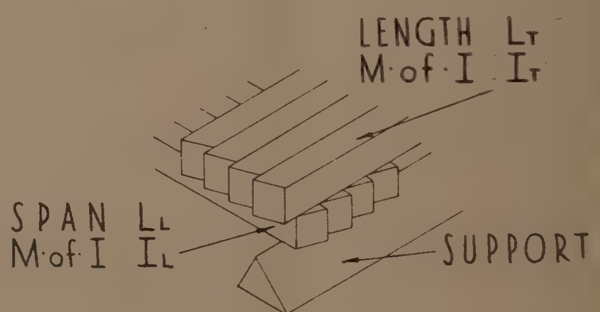


Fig. 2

I_L and I_T are the moments of inertia of the slab/unit length, about axes perpendicular to the sides and supports respectively.

All loading placed on the top beams will be transmitted by reaction through to the lower beams and thence to the supports.

It will be seen that the transverse beams may therefore be considered as resting on a supporting medium whose elastic constant is the relation between the deflection of the longitudinal beam and the force causing that deflection, which will depend both on the degree of end fixity involved, and on the position over the span of the transverse beam.

Taking the case of a transverse beam at the centre of a simply supported span, equations (1) and (2) may now be applied in order to solve for "$d\ max$" and "l" and so find the maximum design moment.

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* *Precis of a paper which is available for reference in the Institution Library.*

Three cases of loading have been considered.

Case 1 : Load at centre span $l < L_T$

Case 2 : Load at centre span $l > L_T$

Case 3 : Load on extreme edge of slab.

The deflection due to the segmental loading was in each case found by using the Mohr Moment Area method.

CASE 1 : LOAD AT CENTRE SPAN $l < L_T$

This applies to wide slabs, there the effect of the load is not felt towards the edges of the slab, and the deflected transverse section assumes the form (Fig. 3).

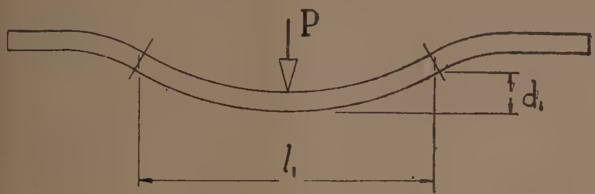

Fig. 3

For simplicity, the effect of end fixity on the length "l_1" is ignored.

This will not affect the accuracy of the result to a very large extent as the degree of fixity is not perfect, and therefore the resulting positive moments will be only slightly greater than those obtained with fixity included.

Similarly any negative moment would be less than that calculated later when a point load on the edge of the slab is considered.

Under these conditions the equations are :—

$$l_1 = \sqrt[4]{5.625 \frac{I_T}{I_L} L_L^3} \quad \ldots \ldots \quad (4)$$

$$d_1 = \frac{5.3\, P\, l_1^3 \times 10^{-3}}{EI_T} \quad \ldots \quad (5)$$

$$M_1 = 0.09724\, P\, l_1 \quad \ldots \ldots \quad (6)$$

CASE 2 : LOAD AT CENTRE SPAN $l > L_T$

In this case the entire slab width deflects giving a deflected shape as in (Fig. 4) and the equations become :

$$d_2 \left[0.302 + 4.01 \frac{I_L L_T^4}{I_T L_L^3} \right] = \frac{P}{8E} \left[\frac{L_T^3}{I_T} - \frac{L_L^3}{6 I_L L_T} \right] \quad \ldots \quad (7)$$

$$y = \frac{P L_L^3}{48 E I_L L_L} - 0.698 d_2 \quad \ldots \ldots \quad (8)$$

$$M_2 = 18.347\, EI_T \frac{d_2}{L_T^2} + 6EI_L\, y \frac{L_T^2}{L_L^3} \quad \ldots \quad (9)$$

CASE 3 : POINT LOAD ON EDGE OF SLAB

The procedure adopted in treating this condition is exactly the same as in cases 1 and 2, and the equations are :—

$$l_3 = \sqrt[4]{0.594 \frac{I_T}{I_L} L_L^3} \quad \ldots \ldots \quad (10)$$

$$d_3 = 0.116 \frac{P l_3^3}{EI_T} \quad \ldots \ldots \quad (11)$$

$$M_3 = 0.245\, P l_3 \quad \ldots \ldots \quad (12)$$

Procedure to be Adopted in Design

The value of "P" to be used is that point load, or equivalent point load on a unit length which is likely to be the maximum imposed on the slab.

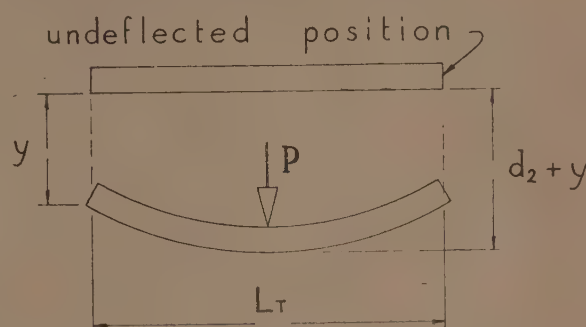

Fig. 4

Having established "P" a check in equation (4) will show which loading case, 1 or 2, is to be used in finding the maximum positive moment.

Since the value of "M" is in inverse relation to the value of "K," it follows that the transverse reinforcement may be reduced towards the supports, and it is recommended that, in the case of simply supported slabs, the reinforcement be reduced at quarter span to 60 per cent. its maximum value.

Book Review

Cornish Engineers, by Bernard Hollowood, illustrated by Terence Cuneo. (Camborne, England : Holman Bros., Ltd., 1951.) 96 pp., 10 in. × 8 in. (Private Circulation.)

This book is published for and is primarily a history of the 150 years' life of Holman Bros., of Camborne, Cornwall.

It gives an interesting history of the family business, its work with men like Trevethich, on the early Cornish engines and the locomotive, and to Government contracts in the war years and projects all over the world in the post-war period.

The firm grew with the metalliferous mining industry of Cornwall and expanded abroad with the Cornish miner to such an extent that despite the decline of the local industry it still has a world reputation for mining machinery and other engineering products grown from the intensive research and careful workmanship required as mines deepened and industry spread.

The interesting history of Cornish mining describes how the high pressure steam engine, the air compressor and pneumatic tools not only saved it several times but allowed the firm to expand into the civil engineering and other industries, where the use of air allowed tools to do work and be used in positions and under conditions practically impossible with other power units. The ventilation of underground working was helped, flexibility of power units allowed work to be done far from static power plants, and pneumatic force was applied in such different spheres as drilling holes, clearing up dirt, compacting concrete and firing projectiles.

Work done in peace and war, helped by the family spirit giving close contacts with both customers and employees, is a fine example of the Cornish motto : "One and All."

S. J. C.

Faults in Concrete Structures

Discussion on Mr. P. G. Bowie's Paper*

The President, proposing the thanks of the meeting to Mr. Bowie, said it would be appreciated that in the presentation of his subject he had given information additional to that contained in the paper. The purpose of the Institutions' meetings was to pool knowledge, and Mr. Bowie had helped in that regard in a very able manner. The hearty thanks of the Institution were due to him, not only for the information he had given, but also for the very able and interesting manner in which he had presented it.

(The vote of thanks was warmly accorded, and Mr. Bowie briefly expressed his appreciation).

Mr. A. H. Ley (Associate) said how very much he had appreciated Mr. Bowie's most enlightening talk, and asked if he could give some help on a question which was always cropping up in private practice, in connection with old buildings. He mentioned a case where there were three separate reinforced concrete buildings, six storeys high, all of which were erected by different contractors during the first World War. They were of similar construction and of very similar design, and in every case spalling had occurred in the beams and in the columns exactly as had been illustrated by Mr. Bowie. Mr. Ley's firm had carried out some experiments to ascertain the best means of overcoming the trouble, and they were given the opportunity to leave the experimental specimens for twelve months before tackling the job in a permanent manner. Various means were tried—the blown-on method, shuttering and pouring new concrete and also what was more or less plastering. There was a marked difference in the final results ; where the old concrete had been cut away, new form work put up and new concrete cast on, there seemed to be very much better adhesion than in the other cases. He asked if Mr. Bowie considered that in the other cases there had been bad workmanship, or whether the method which had given good adhesion was the one that he would recommend.

Mr. Bowie replied that he would recommend the new formwork and concrete method if it were possible to apply it, but this was not always possible ; where there was a large flat surface it was not a very easy method to adopt. One of the older bridges in London led on to Cory's wharf at Purfleet, built in 1907 or thereabouts. It was in a very good state of preservation, though there had been some slight exposure of aggregate. It was a girder of the vierendeel type, and 20 years ago some of the verticals had been damaged by lorries ; it was repaired by the method Mr. Ley had mentioned ; the repairs were perfectly sound to-day. A similar type of repair was being carried out on the main jetty by an experienced contractor, and Mr. Bowie believed it would be a most satisfactory job.

A great difficulty was to secure contact ; we could not get an adequate key without going to a lot of trouble. In order to achieve it one would probably have to cut down to the reinforcement. The provision of shuttering and the ramming of the concrete was the best method, where it could be applied.

Where there was a big projection, one should cover it with lead or equal to keep out the water ; in the case of a minor projection such as a window-sill, one should slope the top.

Mr. C. V. Blumfield (Member), who added his appreciation of the paper, said that nearly all the trouble experienced with concrete seemed to be due to the infiltration of moisture ; there might be a crack in the concrete due to the tension in the reinforcement, and there was not much that one could do about these.

He asked if Mr. Bowie could give an opinion with regard to the various proprietary preparations which could be used in concrete in order to make it more dense and to prevent the infiltration of water, whereby presumably the leaching effect where the moisture came through the concrete and brought out with it the ingredients, could be avoided.

Mr. Bowie commented that he had some little difficulty in giving an answer. He preferred cement to be pure and undefiled. However, perhaps Mr. Blumfield and himself could have a little chat about it in another place.

It was said that the enemy of concrete was water, either when the concrete was mixed, or at a later stage, and he urged that one should use concrete in as dry a state as possible.

Mr. A. W. D. Marshall (Associate Member) said that the members were grateful to Mr. Bowie for having presented his paper on the defects of concrete, for most of them were only too conscious that there were defects.

He suggested that we were not learning from the defects we saw, but were making matters worse for ourselves by being too " clever," and storing up trouble for posterity. He believed that nearly all Mr. Bowie's examples had shown that it was lack of concrete cover that was the greatest cause of the defects.

We should see what the Institution and other technical bodies concerned could do to improve the quality of the concrete and workmanship on the job. It was said that the enemy of concrete was water, and that was true to some degree, but in Mr. Marshall's view lack of water was equally an enemy ; proper consolidation was very difficult to achieve to-day, no matter what was in the specification. When the reinforcement bars had very poorly compacted concrete around them, one wondered whether the adhesion of the concrete around the bar was such as to develop the bond stresses.

Asking how old concrete should be before a protective coating was applied to it, Mr. Marshall said that if a reinforced concrete structure were painted with a substance to prevent water and air getting into it, that substance must also to some degree prevent the drying out of the structure.

Mr. Bowie, as a member of the Institution of Structural Engineers, said he felt entitled to say that one of the main causes of trouble in columns was the architect. He felt that they did push us rather hard in regard to the size of columns. Stilts did not appeal to him ! He was quite content with the concrete covering normally allowed for the inside of a building, but on the outside he would not stop at 2 in. ; rather would he put on 3 in.

Presented at a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, March 13th, 1952, Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair. Published in THE STRUCTURAL ENGINEER, Vol XXX, No. 3, p. 51.

if he could afford it. We put $4\frac{1}{2}$ in. of concrete around a steel stanchion, and he felt it would be a good scheme to apply a very much thicker covering than the normal to all external columns.

The problem of the instruction of craftsmen was one of the matters which the Institution had had in mind for some time. There was an Association which was producing, or was hoping to produce leaflets in that connection ; such leaflets were very difficult to draft, for we must neither insult a man by assuming he was too ignorant, nor talk too far above his head.

As to how long a period should elapse between completing a structure and applying a protective coating to the concrete, he did not think there was any particular limit of time. When using ordinary cement he believed the chemical reactions were completed in a month, and he had in mind applying the dope after that sort of period. But he did not know that one would do much harm by applying it at once. In that connection he mentioned an aerodrome at which a sprayed membrane covering was applied to the concrete paving immediately it was completed. The old idea about concrete was not to let it dry out, but what we wanted was to prevent the water evaporating. He did not think there was any reason to delay the application of a protective coating.

Dr. T. P. O'Sullivan commented that Mr. Bowie's paper was extraordinarily appropriate at the present time, drawing attention to the various factors arising in connection with faults in concrete, particularly in view of the difficulties encountered in obtaining licences for the erection of new structures, more and more engineers nowadays were being required to effect repairs to buildings which before the war would have been demolished and replaced by new ones.

Discussing a problem with which he was faced concerning a large factory, some 350 ft. long, in a mining area, he said the building had settled in such a way that the floor at one end was 4 ft. 6 in. lower than at the other. Dr. O'Sullivan indicated the position of a fault in the ground, the point to which the coal workings had been brought, and the shape which the settlement was tending to take. It was expected that within the next ten years another 18 ft. depth of coal would come out, and that there would be further subsidence of anything up to 10 or 12 ft., but the client still insisted that the building must be kept going. There were important processes which they did not want to interrupt, and they also realised that there was little possibility of getting a licence for a new building.

It was interesting to note the way in which the failure had occurred. As a result of the coal being taken out, there was a draw in the direction from which the coal had been worked. It had been described by a Committee of the Institution of Structural Engineers, which Committee had reported on it in a paper published by the Institution.*

It was a credit to the designers that, in spite of the differential settlement, by reason of which one half of the building was of mushroom construction, and the other half of beam and slab construction, the floor was almost unimpaired, there being hardly a crack in it. The factory was built in 1925.

Dr. O'Sullivan went on to refer to a road failure, one of the most spectacular he had ever seen. There were expansion joints at every 60 ft., the slabs were 6 in. deep and the jointing material was only 4 in. deep. It was found eventually that in effect there was no expansion joint over a length of about 1,500 ft. Three days after the casting of a certain section a failure occurred, at one point the surface being raised 2 in. It was one of the most sudden failures he had ever seen, and he did not want to see such a failure again.

Mr. J. Owen Lake (Associate-Member) said Mr. Bowie had mentioned that a point at which cracks were frequently obvious was in external columns at floor level, and he had attributed that to the presence of construction joints, congestion of reinforcement, the combination of cranked and spliced column bars and the prevalence of high bond stresses. Cracks had also been attributed to moisture and temperature movements.

Mr. Lake drew attention to a factor which might often be a more fundamental cause of cracking in external columns at floor level, namely, settlement. In designing steel frame or reinforced concrete frame structures it was almost invariably the practice to assume that they would be erected on rigid foundations. If the effects of even quite small differential settlements on the bending moments and shear forces in a framed structure were analysed, an unsatisfactory condition was disclosed. For example, Meyerhof† in an analysis of typical building frames, had found that a relative settlement of adjacent bases of less than $\frac{1}{2}$ in. almost doubled the initial stress calculated with no settlement at critical sections in beams and columns. It was perhaps significant that the critical sections where the greatest increase of secondary stress occurs were the junctions between external columns and beams, especially in the lower storeys of a framed building. It would appear, therefore, that the neglect of the influence of settlement on superstructure design might often be the primary cause of such faults.

There was a tendency, continued Mr. Lake, to advocate higher working stresses, and inevitably that necessitated further refinement of structural analysis. However, such calculations based on the assumption of rigid foundations would have no validity whatsoever if such a concept was not realised in practice by properly designed foundations.

Mr. Bowie said he had merely stated his views on the matter. In a normal building one column might be loaded and another was not ; he doubted very much that in any building of average size all the columns had settled to the same extent. They had all settled, there might be 2 in. settlement anyway, and whether it was 2 in. or $2\frac{1}{2}$ in. would theoretically make a lot of difference to the stresses, and he did not think that by analysis we should get much closer. Of course, if one feared something of that sort, one could modify a building. By using a mushroom slab one would get a much more flexible floor structure, and consequently less stress would be imposed on the column due to settlement. In columns, not only did one have a congestion of bars, but often on the next storey the column was not quite where one had thought, and it became necessary to cramp the bars somewhat. Such things led to the introduction of secondary stresses due to the kinks and bends in the bars, which stresses led to cracking. There was a tendency in the United States to avoid that cracking and the lapping of column bars by welding on the next lift. The old idea was to use a piece of gas barrel in order to join two lengths of bar.

If we looked at a column and followed the normal methods of calculation we could see there was a change from compression in one face of the column to tension on the same face below the floor ; therefore, we must have very high bond stresses in the bars that ran right through.

**Report on Mining Subsidence and its Effect on Structures,* 1950. 5s.

†*The Settlement Analysis of Building Frames,*" by *G. G. Meyerhof.* THE STRUCTURAL ENGINEER. *September,* 1947.

Mr. C. B. Brown (Member) said that he was particularly struck by Mr. Bowie's reference to reinforced concrete structures which had failed at about the age of 25-30 years. Admittedly, there were quite a number of reinforced concrete structures of considerably greater age—he believed that in this country their ages ranged up to about 50 years or even more—which were still in perfectly sound condition, and had not required very much maintenance.

Referring to a job which had been quoted by a previous speaker, a warehouse which had cracked up very badly, Mr. Brown said he believed it was built about 30 years ago, and the main cause of the failure seemed to be bad concrete. There were other faults; for instance, the design had left much to be desired, and the structure had been overloaded. In the course of investigations on that job some enquiries had been made of a number of people about failures in concrete. The general opinion was that when reinforced concrete construction had first started in this country it was undertaken by firms of high standing, who had carried out a very high standard of work, and the results in general had been very good. However, when the "concrete rage" came on, which he believed was just after the 1914-18 War, a good many mushroom firms had grown up, and everyone who had thought that he knew anything about concrete had immediately launched into the concrete contracting business. Thus it was possible that during the 10 years or so following 1918 quite a number of very indifferent structures were built in this country. Mr. Brown believed that the main fault in those structures was the use of bad concrete. There was at that time a good deal of price cutting which, naturally, was reflected in the amount of cement that went into the concrete.

One would like to hear whether Mr. Bowie considered that theory to be correct.

Mr. Brown also added his appreciation of an extraordinarily interesting lecture.

Mr. Bowie did not think that he could deal with that matter completely. He quite agreed, of course, that the original reinforced concrete structures were usually erected by contractors who were licensed by the designers; in those circumstances there was good supervision of the work. But then we had a war, and by the end of the war we had forgotten a great deal of what little we had known about concrete. He himself had had a shock when he came back after the war; some plans were handed to him and he had to hurriedly try to remember what he had done five years previously. He supposed the contractors were in the same position. Then anybody who had thought they could make concrete came into the business; the technique was rather poor, and he believed that not so much trouble was taken then as formerly. He did not think the Government contractors and engineers had quite realised how bad concrete could be if the work were not supervised adequately. He had seen the concrete at roof level in a five-storey building of almost entirely 2 in. aggregate, which had obviously been scraped from the bottom of the heap; the contractors had simply used it up. We might not be perfect to-day, he said, but the work nowadays was a great deal better than formerly.

Thus we had experienced the two extremes. At first the contractors were worried about reinforced concrete construction and later they had become over-satisfied with it; and nowadays he believed we were getting back to the stage at which it was getting the attention it deserved.

Dr. B. H. Knight (Member) mentioned one or two points which bothered him, though in some cases they might appear to be elementary.

Fig. 1 in the paper showed a bomb-damaged building, and he would have described that as a testimony to concrete; he asked if Mr. Bowie would agree with that view.

With regard to Fig. 2, which showed Limonite staining on a concrete surface, he said that as a rule it started in a slightly different form; it started with marcasite in the pits from which the stone came. In concrete it took 7-8 years for the staining to appear; so that it was a slow process which he believed was due to hydration and possibly to alkaline reaction as well. He did not think there was a remedy for it expect to cut out the offending stone and start again. He had tried to find a means of dealing with the problem in the pits. but that was not easy, and it seemed that the only thing to do was to get to know the pits which produced that stone and to avoid the use of aggregates from those pits.

Being unable to understand the captions to Figs. 4 and 5 in the paper, Dr. Knight asked if Mr. Bowie would say what they meant.

On page 53 there was reference to the effect of heat on concrete structures, but nothing was stated there about the work done recently by the Building Research Station concerning the expansive properties of concretes made with different aggregates. A lot of buildings had to be constructed with siliceous aggregates. Dr. Knight asked if the author could amplify his remarks on the effects of heat, and could indicate how he dealt with the matter in order to avoid trouble due to the expansive nature of the materials.

One of the worst features of concrete was its bad appearance if it were not well placed, and he asked if the author would agree that methods of placing concrete were somewhat primitive. It seemed, however, that in road work the matter had been studied a lot. Some people advocated the use of precast members in buildings, but it would be interesting to hear Mr. Bowie's thoughts about ways of putting concrete in formwork over large areas which would give it a nicer appearance and would render unnecessary the use of bush hammers, and so on.

Mr. Bowie, referring to the bomb-damaged building in Fig. 1, said there were companion photographs showing how considerable was the damage to the outer wall and how well it was repaired. The building had suffered a very considerable shock, which had jumped the columns right off their seats, so that the splice bars were forced out. The photograph was intended to illustrate the great resistance afforded by reinforced concrete.

As to limonite staining, it did not appear to be very difficult to chip out the offending material and refill by pushing in a pat of mortar and a stone.

Coming to Figs. 4 and 5, he said the photograph in Fig. 4 showed an example of the effect of frost on a concrete which had very fine material in it. It showed part of a bridge abutment. There was stone dust in it, and where high proportions of dust were used there was a corresponding degree of porosity.

Fig. 5 was a photograph of a bridge parapet which had a flat top and on which water collected. In parts of Scotland the rainfall was as much as 200 in. a year, and the top of the parapet became absolutely saturated. If we saturated anything and then froze it, something had to go, and in that case the concrete at the top had come off.

Dr. Knight commented that it made a difference whether one used broken brick or sand ballast or some of the limestones.

Mr. Bowie agreed that that was so, and said that quite good concrete was used in each case. If we had to use a particular cement with each particular aggregate

it would entail disadvantages from the commercial point of view. Dr. Knight referred to the effect of temperature on aggregates, and he agreed that with extreme ranges of temperature that factor would become important, but only where there were extreme or abnormal ranges of temperature, such as the flue range or the fire testing range. At temperatures of 40°—60° or something of that sort he did not think that the difference between aggregates caused much trouble.

Mr. S. R. Broderick remarked that he was afraid he might experience a sleepless night, for a speaker in the discussion had stated that water was bad for concrete. As one who dealt with water-bearing structures, he found that water improved concrete. For example, recently he had inspected a waterworks with a view to its extension. The works had been in use for 26 years; the filter bed structures were of good concrete, and although there were a few cracks, in general the concrete was in good order. He had seen the original drawings of the structure and had been astonished by the small amount of steel used. He considered that the stresses in the walls must have been very high and, therefore, a certain amount of cracking should have been expected, leading to rusting of the steel, followed by spalling. However, on a building which was erected at the same time as the works, there were points at which the concrete had spalled from the reinforcement.

Mr. Broderick was of opinion that concrete surfaces under water or partially under water were much more lasting than surfaces on the outsides of buildings. He had to build water towers, which were most difficult structures in concrete, for there were so many stresses to be dealt with. One side was exposed to the heat of the sun and the opposite side was in shadow, and one would expect cracks to occur all over the place, but the towers he had put up did not appear to suffer in that way. Undoubtedly the basis for lasting structure was good concrete.

Mr. Bowie said he was speaking rather generally when he had said that water was an enemy of concrete. It was advisable to keep the proportion of water as low as possible in a mix, provided the mix was not so dry that it would not consolidate properly. But he felt that to some extent the proportion of water that could properly be used in concrete depended very much on the structure. For a road or paving one could use a very dry concrete, because one could hammer it down; thereby one produced a better paving, being able to ensure adequate consolidation.

Expressing sympathy with water tower designers, Mr. Bowie said that the towers lasted very well. He mentioned a case in which a tower became oval between dawn and sundown. Designers had to think about that, as well as the use of asphalt linings.

If steel caused trouble we should not put in so much. The use of a deformed bar would probably lead to less harmful cracks.

A speaker said that from Mr. Bowie's last remark it would appear that steel in concrete was largely the cause of its misfortune. The parent stone, which we tried to simulate with concrete, beautified with age, and it had no steel in it. He suggested to Mr. Bowie that there was a tendency amongst designers in reinforced concrete, by virtue of their being reinforced concrete designers, to say that reinforcement was necessarily of importance, even in the lesser stressed members. There were many subsidiary members in concrete structures which would do their job perfectly effectively if they had considerably less reinforcement, or none at all.

Mr. Bowie, remarking that he had really let himself in for it, said we should not put in reinforcement unless we considered it to be necessary. The amount of binding that was put into cornices and projections of that sort was often unnecessary. It did not add to the strength; the concrete would crack before the reinforcement was stressed.

We wanted to ensure that bars were not too near the surface. The difficulty was that, in placing and punning the concrete, the bars were liable to be moved. A system he wanted to try out was one using wire loop in a half sphere of concrete.

Mr. J. Singleton-Green (Hon. Curator) said that although the discussion had been concerned mainly with deterioration on the outside of reinforced concrete structures, he would refer to a floor job which emphasised one of the points made by the author. The floor was in an office at a gasworks. Those responsible had gone to a lot of trouble to produce a white floor; they had used white cement and a white aggregate. When the floor was laid it looked a splendid job, but on the following morning the whole surface was a dark brown colour. It took several days to solve the problem, but eventually it was found that there was a leak from one of the pumps, and H_2S had escaped into the room in which the floor was laid. That would not have mattered but for the fact that in the Derbyshire spar there was some "dust," and in the dust there was lead. The H_2S had reacted with the lead and formed lead sulphide.

Another example which was rather startling concerned a foundation slab for a small furnace used for treating copper. The furnace dropped 6 to 9 in. and tilted. When the furnace was removed, it was found that the concrete in the foundation had melted. The heat in the furnace was not insulated sufficiently by the bricks; it had got through to the concrete, which had melted, and then solidified to glass of a dark green colour.

There were two outstanding points in connection with the subject under discussion. One was that we should put durability before strength; but unfortunately there was no "yardstick" for durability. If we wanted to measure durability we had to do it to some extent by using the cube strength.

The other point was control of the concrete on the job. He had urged job control for many years and had suggested that there should be concrete "craftsmen." It was a big problem, but some day it would be solved. It was complicated, he thought, not from the technical point of view, but rather by trade union consideration.

Mr. Bowie said a trouble sometimes met with was that there was a drying up of the foundation underneath a heated slab, so that quite a hollow was formed under the floor and settlement occurred. But that was a minor difficulty compared with the case mentioned by Mr. Singleton-Green.

The President, at the conclusion of the discussion, said it would be agreed that some very interesting comments had been evoked. He was rather surprised, however, that there had not been more discussion on deformed bars. Some 30 or 40 years ago they became quite popular, but their use seemed to die out in this country on the grounds of first costs and possibly as the result of by-laws and regulations. Now they seemed to be coming into prominence again, and we might see much more of them.

Finally, on the President's proposal, the meeting expressed thanks to Mr. Bowie for the able way in which he had dealt with the discussion; and he expressed his appreciation.

Unusual Design for a Large Constructional Shop*

Discussion on Paper by F. R. Bullen

Mr. Bullen exhibited a scale model of the building described in the paper, and demonstrated the features discussed therein. He commented that the great advantage of such meetings was that the views of other members of the profession could be obtained. Being an art as well as a science, structural engineering was often capable of more than one solution and consequently designers could benefit by the comments of their colleagues.

The President, proposing a vote of thanks to the author, said that the Institution owed a debt of gratitude to Mr. Bullen and to his colleagues for having presented such a paper. It was a very useful and practical addition to the Proceedings, and the members would wish to express thanks to Mr. Bullen for the interesting and fluent way in which he had presented the paper.

(The vote of thanks was accorded with acclamation).

Mr. Bullen briefly expressed his appreciation.

Mr. H. C. Husband (Immediate Past-Chairman, Yorkshire Branch) said he need make no apology for joining in the discussion. Mr. Bullen would appreciate the genuine interest with which he had read, and re-read, the paper if he mentioned that in his office a set of rough calculations had been prepared to fit the design described. He had compared the structure with a two-hinged portal frame building of 80 ft. span with main frames, for planning reasons, at nearly 70 ft. centres, which his firm were putting up, and he could say straight away, in view of the claims made in the paper, that the steel economy achieved in Mr. Bullen's structure was appreciable over the normal portal frame design, if one compared designs from the foundation upwards.

Mr. Bullen should be particularly admired and thanked by the profession as a whole for having produced a design so far out of the usual ruts and, moreover, a building having such delightfully clean lines. Mr. Bullen had commented upon the addition of cables and services to the building, but even these had not been able to spoil a really beautiful building.

A matter which had impressed one immediately—and the author had drawn attention to it in the paper—was that of transferring the horizontal crane forces directly to the ground through the tapered cantilever columns. The ground conditions were apparently not particularly good, and piling was employed. He asked if the author could give an indication of the increased cost of foundations in order to obtain sufficiently complete fixity to the columns over that which would have been the case had portals hinged at their feet been employed. Was consideration given to the overall cost of the structure, when the cost comparison was made with the first design on the basis of continuous frames?

It was stated in the paper that the possibility of hinged joints at the ground level had been investigated. It would seem hardly fair to claim that "it was more logical to transfer the crane forces direct to the ground through the columns" unless the ground was good for providing fixation. Did the author consider the alternative of roof portals above the caps of the crane stanchions, possibly with pin joints at the stanchion caps? That alternative would appear to give a slightly lighter roof, perhaps 16 × 6's throughout. A greater outward thrust might require to be dealt with at the outer columns, but crane gauge control might have been slightly improved.

The articulation suggested the possibility of unequal settlement, and Mr. Husband asked whether that was a factor which had influenced the design. If any settlement did take place, might it not lead to more crane gauge variation than with a rigid portal connecting the crane girders?

The horizontal crane forces had been concentrated on the two main lines of cantilever columns by the use of sloppy fits on the remote crane wheels. That suggested that the wear on the near crane rails would be greater than normal, thus decreasing their life. The fastening and replacement of heavy crane rails was always an interesting detailing problem, and Mr. Husband wondered how those rails were fastened.

It was to be noted that the 3 in. by 3 in. billet rails in the longitudinal bays were tack welded to the girders. He understood that quite a lot of trouble had been experienced at one of the New South Wales works in connection with that method of attachment, and he wondered whether any snags had been experienced on Mr. Bullen's job.

Reverting to the matter of the sloppy fits on one set of crane wheels, he wondered what allowance was made for lateral crane forces being transferred through the wheel friction on the loose fitting side.

In the case of the longitudinal bays, did the author consider the use of rigid frames for those single spans? It appeared that the roof there was not lighter than would be the case with a rigid frame, since there was no continuity at the eaves.

On page 61 it was stated, in reference to the transverse bays, that the balanced ribs consisted of one fixed apex and one sliding apex. Mr. Husband asked if that were correct and, if so, what type of sliding connection was adopted.

Asking for the author's views on provision for crane maintenance, he said that in the transverse bays there appeared to be a reasonable walkway at crane rail level; but that was not so in the longitudinal bays, where there appeared to be no facilities for servicing the two 50-ton cranes. In neither type of bay were arrangements shown for high level servicing of the cranes or the lifting tackle over them. Those were provisions which were becoming increasingly popular.

The author had mentioned that the erection of the building presented rather more difficulty than usual owing to the design. Mr. Husband asked if he could

**Read at a meeting of the Institution of Structural Engineers at 11 Upper Belgrave Street, London, S.W.1, on Thursday, March 27th, 1952. Mr. Walter C. Andrews, O.B.E., M.I.C.E., President, in the Chair. Published in* THE STRUCTURAL ENGINEER, *Vol. XXX, No. 3, p. 55.*

[g]ive the overall cost per ton of the steelwork, including [e]rection costs ?

Did the saving in weight, as compared with a more [c]onventional and perhaps more easily erected portal [d]esign, together with the simplification of fabrication, [p]roduce an overall financial saving ?

Finally, he asked whether crane surge tests had [s]hown that the lateral stability had come up to ex[p]ectations. He wondered whether, on the completion [o]f the job, actual deflections had been compared with [t]hose calculated. The question of overall rigidity was [v]ery important.

Mr. S. F. WARBURTON said he had listened to Mr. Bullen with a great deal of interest and a desire to know [m]ore, and he expressed thanks to him for all the industry [a]nd judicious sorting of data required to give the in[f]ormation contained in the paper.

The first information for which he asked was the form [o]f articulation at the valley columns to prevent moment "carry over" to the rafters ; and the provision for [s]ervicing the cranes (i.e., lifting the crab, etc., which in the case of 20-ton cranes would mean handling a crab of (say) $4\frac{1}{2}$ tons.)

There were two major points, which it had been inferred were fundamental, on which Mr. Warburton joined issue.

The first was the design approach, mentioned in Section (1), "Transverse Bays." The main function of the steel skeleton, he understood, was to accommodate and support the dynamic loading from the cranes and the jibs. How, then, could an increase of general stiffness by fixity of eaves and valleys be the "fundamental error" ?

Those who had a working knowledge of the maintenance of cranes on outside gantries, with every degree of freedom at gantry level, and of the punishing effect of long-armed jibs on gantry stanchions, might agree with his contention that maximum rigidity was essential to the well-being of a gantry building.

If the total strain energy in the steel skeleton must equal the external work done, he found it hard to believe that Mr. Bullen considered successive degrees of freedom, with a consequent increase of stress and elasticity concentrated at fewer points, to be fundamentally correct.

The "error," defined by the author, of direct loading and bending was inherent in the side stanchions of the transverse bays, with bracketed gantry girders. It appeared that the author had taken the bending from the knee and had put it into the stanchion and rafter ridge.

If the gantry shaft had been placed under the gantry, that would have been obviated and the "carry over" from the side surge would have been small.

It was possible that that arrangement was dismissed because of the small increase of floor space required by the stanchion, but it appeared as though the bases of the two valley columns were wide in consequence.

His second point concerned the "fundamental" of economy. In the light of his own investigations into various types of buildings with and without fixed feet, he doubted the author's saving of 10 per cent. in cost over the orthodox rigid frame, due to the novelties that were introduced. For those comparisons Mr. Warburton found it essential to view the cost of steel, erection and foundations in total.

It was true, he said, that fixity at the eaves and valley forced those points to accommodate "carry over" moments, but if the lowest section of the gantry stanchion could be proportioned adequately, those "carry-overs" were rarely of an order to create problems, and that fixity had a definite value in rigidity of the building.

Dr. Hendry had been at pains to show how the inclusion of a haunch provided dividends out of all proportion to the initial cost.

Again, if we assumed *a priori* that stanchions with "rounded ends" showed no saving in steel over stanchions with fixed ends, it was pertinent to ask where the economy occurred. If it be in the roof, then questions of stress and flexibility were involved, as it was a gantry building with a crane gauge to maintain.

It was also true, as the author had said, that the rafters were provided primarily for keeping out the weather. But they demanded a certain minimum section for deflection, and why should they not also be made to work for a living up to the limits they first required for stiffness—but in another part of their length ?

To fix both ends of the rafters certainly produced problems in analysis, but it did not increase the total value of moments in the frame due to dead and super loads. What the author had accepted at one point, and provided a compound member to accommodate it, might be taken at several points within the compass of a rolled section.

Mr. Warburton showed two slides to illustrate his points. SLIDE 1 indicated the plan and two typical sections of a building similar in function to that designed by Mr. Bullen. The plan area was 170,000 sq. ft. approximately. Nearly all bays had cranes in tandem and in one case cranes in tiers. All stanchions were designed for jib loading. All members were rolled steel sections.

With one main exception, there was no "carry over" of moment from vertical gantry and jib loads into the roof structure, since the bases were fixed, supported on piles, and gantry girders were directly over shafts.

Only lateral live loads, therefore, affected the roof framing. At the instruction of the clients, the framing was designed for the worst combination of any loading without reduction in side surge loading due to cranes in tandem. Excluding wind forces, about 90 per cent. of the sections were within 8 tons/sq. in. in bending.

In Mr. Warburton's opinion, without the compensation of the main ties of the orthodox trusses it would have been an error of judgment to have introduced articulation into those frames for the saving of 10 per cent. of steel to use Mr. Bullen's approximation.

SLIDE 2 was a section of the four-bay frame, chosen because it had fewer members to absorb any transverse loads and on stanchions *B* and *C* had the greatest surge loading acting at the highest level.

It would be noted, as Mr. Bullen had suggested, that this affected all members ; but also, as Mr. Warburton had mentioned, the ratio of the "carry over" moments to the dead and super loading moments shown was very small.

In the particular instance of Mr. Bullen's design the ratio would have been still smaller, in view of the larger spans of 85 ft. 6 in. and the increased roof loading.

The introduction of haunches at the points of rigidity also reduced the stresses in the members from the maximum shown at the theoretical intersection points.

In conclusion, Mr. Warburton apologised for having occupied so much time, but said he felt that for the sake of all the younger designers some qualification of Mr. Bullen's statement with regard to his "fundamental" design approach should be made with respect to gantry buildings.

With the B.S.S. 449 rather broad guidance on gantry loading and its apparent blessing of high stresses for

them, it was well to remember that a lot had still to be written about dynamically-loaded structures. Many a maintenance engineer, worried about his cranes, little realised that the trouble started in their flexible gantry supports and girders.

Mr. E. M. LEWIS said that after a previous paper by Mr. Bullen he had been thanked by the author for one or two ideas he had put forward to do with model analysis. Perhaps on this occasion Mr. Bullen would reciprocate by answering a number of specific points concerning the problem of gantry crane rail fixing.

The paper referred briefly to the use, as a rail, of "Three-inch square steel billets tack welded." Mr. Lewis assumed that in fact these were attached by pairs of substantial intermittent welds capable of transmitting the shear forces engendered by the action of the rail as an integral part of the top flange of the girder.

There were a number of points about the welding down of such rails.

(a) Were they rolled to special profile and if so what size were the "ears" by which they would presumably be welded to the flange?

(b) Assuming they were of a rail quality steel, what welding procedure was used—what type of rods, what current, how many passes, was a moulding block used, etc.? Were there any teething troubles?

(c) Were the billets found to be free of twist, if not what was done about it, or, if nothing was done, how much twist was found tolerable?

(d) If mild steel billets were used, the welding would have been straightforward except perhaps for point (c); but if this was the case, what intensity of crane wheel pressure was worked to, i.e., what were the axle loads, the diameters of the crane end carriage wheels and the widths of the rails?

In the discussion Mr. Husband had queried the advisability of welding down such rails. Mr. Lewis said that while admittedly there had been some teething troubles with this relatively new method of fixing, these had by now been largely overcome and in his view current experience was indicating that welded-down rails might well prove to be entirely free from the chronic troubles so often experienced with the various forms of clip used hitherto. With these it was very difficult to make the rail work integrally with the girder flange and the consequent relative movement would seem almost inevitably to lead to a vicious cycle of wear—play—more wear.

With regard to Mr. Warburton's comments about the effect of deflections of the structure on crane behaviour, Mr. Worthington, a colleague of Mr. Lewis', had carried the work of Diamond and Frankau on the behaviour of cranes with taper treaded wheels, a step further. He had analysed the behaviour of cranes with parallel treaded wheels theoretically and had confirmed the work experimentally by means of models and also by observation of cranes in service. Broadly, his conclusions were that much of the trouble due to skewing was caused by end long misalignment of end carriage wheels and, further, that this was accentuated by the presence of end float or "slop" of the wheels along their axles. Perhaps where cranes were troublesome, too much attention was being paid to the structure and too little to the cranes themselves.

In conclusion, Mr. Lewis said he would like to mention one point which puzzled him in the paper. He was not clear what mode of buckling was referred to on page 59, where mention was made of the use of intermediate stiffeners to give additional support.

Mr. DONOVAN H. LEE (Member of Council), adding to the enquiries by Mr. Lewis concerning the crane rails, asked how the billets were machined and whether they were true to shape. As to the method or arrangement of jointing, he asked where they joined in relation to the joints of the crane girders themselves. He remarked that, no doubt, everyone concerned with the design of crane girders would agree the joint was the vital point in crane rails and with the rails possibly welded to both girders the precise detail used would be quite important.

Mr. M. F. PALMER (Member) commented that the shop which Mr. Bullen had described and illustrated must be the envy of many works managers throughout the country.

He wondered whether, if a greater variety and larger sizes of broad-flanged beam were available in this country, it would have assisted in the design of the job and cheapened the construction. In the United States beams of 36 in. by 16½ in. were obtainable.

He was also interested in the statement that the holding-down bolts were fixed finally into position by templates, and he wondered whether any "play" was allowed.

As to the roof glazing, it was very interesting indeed to read the author's comments that the "pepper-pot" form of lighting showed no advantage in tall buildings such as these. He much preferred the appearance of straight runs of glass for this type of shop.

Where there were a number of bays—in that case four—running parallel with each other and connected to the same stanchions, it was necessary to be quite sure of the foundations, because if there were any possibility of some of the stanchions getting out of "plumb," the crane tracks would become out of alignment and the trouble was liable to extend right across the four bays. This might then be very troublesome to correct.

Finally, Mr. Palmer pointed to the heating of the shops and suggested that a few details on that matter would be useful and interesting.

Mr. E. G. CLARK (Member) added information concerning the fabrication and erection of the structures described, particularly in regard to some of the difficulties which were encountered and overcome.

First, he believed all were agreed that the design was unusual, particularly from the point of view of the number of pin joints. The contractors had appreciated this from the start, but certain erection difficulties arising from the nature of the design, had to be overcome during the progress of the erection.

The fabrication of the job did not present particular difficulties from the fabricator's point of view. The sizes of the pieces were governed primarily by transport arrangements and provision for access to the site. In the first place it was intended to transport the large 22-ton plated crane girders, which were about 85 ft. long in three pieces, because there had been some doubt as to whether they could be delivered to the site in one piece. Finally, however, it was found that complete girders could be taken on to the job, and the three pieces were welded together in the fabricator's shops before being taken to the site.

The method of erection, disposition of cranes, and general procedure at site were the responsibility of the steelwork contractor.

In the course of fixing the steelwork in the North High Bay building, the first problem encountered was the spread of the intermediate rafters. There was a system of channel bracing in the plane of the roof and

[I]n the first bays, an attempt was made to fix this bracing as the erection of the intermediate rafters proceeded. This was found to be unsatisfactory, as, due to the spread of the rafters, it was necessary to lift and support the rafters in order to insert the bracing. In subsequent bays a substantial temporary tie was introduced to hold the ribs in position until the bracing was erected.

These ties were then removed and re-used in the following bays.

Another major problem was the treatment of the horizontal forces in the main longitudinal high bay buildings. These horizontal forces, which Mr. Bullen had mentioned in the paper, were taken up in a system of bracing on the low bay buildings. This raised the question of the stability of the high bay building until the low bay buildings were partially erected and the bracing put in. They had considered the problem at the time of the erection of the North High Bay. The bay was not sheeted at that stage, so that it was not receiving the wind forces that it would eventually receive, and it had also the support of the services building at the back of the high bay; thus it was problematical whether it was necessary to stay the building until the work of erecting the low bay buildings was in progress. However, to ensure the safety of the structure steel wire guys were introduced temporarily to stay the high bay building until the erection of the low bay buildings had proceeded. The guys in the high bay building were introduced at each column from the eaves level to the ground.

There was no doubt that anyone who went into the finished factory would appreciate that it reflected great credit on Mr. Bullen, Mr. Hargreaves and all who had anything to do with the original conception of it.

In reference to the crane girders which projected from the low bay building into the high bays, Mr. Clark wondered whether Mr. Hargreaves had found, in operation, that the projection of the girders into the high bay was in the nature of an obstruction to the operation of the high bay cranes, moving materials in that bay.

Finally, he was very interested to note the two references in the paper to the low average price of the steelwork. He would make no comment on that.

Mr. O. Bondy (Member) said that on reading the paper he was impressed by two aspects particularly, first by the unusual type of design based on articulation, and secondly, by the extremely frank way in which Mr. Bullen had discussed his difficulties as the designer. That frankness would be appreciated by his audience; we heard and read descriptions of important and unimportant buildings from time to time, but we did not always hear much about the difficulties that arose and how they were overcome.

One of the problems was in connection with the separate roof girders and the separate rigid columns, as compared with the continuous rigid frame design. He, too, could not help feeling that the "fundamental error"—those were words which Mr. Bullen had used in the paper—in transmitting the horizontal forces through the rafters was not an error after all. It was one of the outstanding recent tendencies in steel construction and also in concrete construction, to make use of continuity. What the younger generation of designers needed most was to become thoroughly acquainted with the advantages of continuous design, especially in steelwork, in order to save steel and to reduce total cost.

Coming to the reference, on page 58, to the economy in the sizes of the roof rafters and to a form of double cantilever construction, Mr. Bondy asked whether the saving was not balanced by additional cost of excavation and foundations for the cantilever columns.

On the large crane girders the top flanges were 36 in. wide and $2\frac{1}{2}$ in. thick. He asked whether they were of uniform thickness over 85 ft. length.

As to the crane rails, of 3 in. square billets, tack welded to the top flanges of the girders, he said they would take some share of the bending moments. In one other case at least the effect of the crane rails had been neglected in the design of gantry girders, and according to calculations he had made, the loss in economy was well above 10 per cent.

In adding his thanks to Mr. Bullen, Mr. Bondy said that one admired the way in which he had repeatedly brought his experience before the Institution, and one was grateful to him.

Mr. F. M. Bowen (Member), referring to the roof of the building, said that the model showed how closely the series of low-pitched bays approximated to a flat roof and, in view of the large surface area, he wondered whether or not wind drag had had any appreciable effect on the sizing of the members. This appeared to be a case where it might have done so.

He was interested to know that the holding down bolts had been pre-fixed in position because he had used this method himself and believed that, for certain jobs at least, it had advantages over the "traditional" method of grouting in anchor bolts after the steelwork had been erected. He wondered whether Mr. Bullen had had any difficulty in persuading the contractors to employ the former method and whether it had caused any difficulty in practice. His own experience had been that after specifying it and making clear that no variation to post-grouting would be permitted, there had been no trouble.

Another common practice in connection with stanchion foundations that merited attention was that of filling with liquid grout the space between the underside of the baseplate and the top of the concrete foundation. The use of grout, to his mind, was somewhat illogical because it was putting a comparatively weak material in the very position where the pressure was a maximum and also where shrinkage was least desirable. Some years ago he had seen a baseplate cut back to the stanchion face; the grout underneath was found to be cracked though otherwise apparently well done, yet pieces could be pulled out quite easily from underneath the base, thus indicating that one could not be certain of achieving a really solid bearing over the whole area of the base. He personally much preferred to use a fine dry concrete backing rammed in under the base from opposite sides.

Reverting to the building itself, Mr. Bowen supported Mr. Husband's remarks about its appearance. The framework did indeed appear to be a fine example of modern structural steelwork. In association with the appearance, he had noticed from Mr. Bullen's illustrations that the painting seemed to be of a peculiarly light and pleasing nature, and, in view of the fact that there was an industrial consultant on the job, he wondered if the practice (which was becoming more common) of paying particular attention to the correlation of colours in painting the several parts of the structure and its contents, had been applied. In view of the claims made elsewhere that such a painting system stimulated production to a useful extent, this was a subject which would appear to be worthy of further attention by structural engineers.

Mr. R. R. Gardner (Member) said the original scheme, if he remembered aright, was for one bay to be of "Clerestory" design for the first shop. That would

obviously have meant the use of much more steel in the roof. Later, however, Mr. Bullen had come in as consultant, and the finished structure, in Mr. Gardner's opinion, was a much better job.

Commenting that the problem of the jib cranes on the stanchions had perhaps been a little over-stressed, he said the cranes were hand-operated and usually employed in fabrication, for lifting plates, and so on, to the machines, and the loading was usually very light, for otherwise the men would not be able to handle the loads. He believed the attachment to the stanchions was quite common practice. The whole structural layout was of first-class design.

Mr. T. K. Hargreaves said that the structure described had been in use now for about nine months and had proved thoroughly satisfactory. Early in the paper it was stated that the company wanted the most modern type of building. This was not just for the sake of modernity but because they considered that a modern welded structure would be the most economical type. This they felt had been confirmed in practice.

Referring to comments made on the design of the crane beams, with regard to the maintenance of cranes, he said that there had been a few small troubles to start with, but these had been simply overcome and he did not anticipate any difficulties in the future.

Mr. F. R. Bullen, replying to some of the points in the discussion, said he would like to reply to the remainder in writing ; the time was late and the questions were many.

Two speakers at least had referred to the effect of transferring the horizontal loads to the columns and had enquired whether the economies achieved in the steelwork had been at the expense of the foundations. That was a very good point, and it served to emphasise that economy of building construction could not be described in terms of one part of the structure alone ; it must cover the economy of the building as a whole. He would not like to say at the moment whether there was any extra cost in the foundations by reason of the economy achieved in the steelwork, but he rather thought there was not. He had gained the impression at the time the structure was being designed that the approach taken was the most economical, especially in view of the fairly bad ground conditions with which they were faced. He believed the design adopted was found to be the most economical, having regard to the foundations as well as to the superstructure ; but he would look into the matter and try to give a fuller answer in writing.

Portals above crane level were not considered. That question rather struck at the whole basis of the scheme ; they had tried to do away with portals, believing that their elimination was a contribution to economy.

He did not know of any troubles on account of the crane gauge. There had been a little trouble when the roof sheeting was put on, for it had tended to press the external hinged columns outwards, and it was necessary to unbolt and re-fix the crane beam inwards to take up that movement of the structure. It was only a question of introducing packing between the stiffeners and columns. There had been movement since, but it was not appreciable.

No account was taken of the friction between the crane wheels and rails ; it was assumed that, by reason of the so-called sloppy fit, the horizontal forces were taken to the main columns.

The crane rails were just welded to the girders ; they were intermittently welded.

No tests had been carried out on crane surge. There were many people about, and tests were not very welcome usually.

In reply to Mr. Warburton about the form of articulation to prevent " carry over," he said they had tried to avoid those things ; they had tried to design on practical lines. Perhaps that answered Mr. Bondy's question also. Whilst one did not wish to depreciate the theoretical treatment, perhaps he might repeat that economy did not always come from finesse in design so much as from the careful arrangement of members.

Coming to the point made by Mr. Warburton by means of his slides illustrating the buildings he had had to deal with, Mr. Bullen said that in the case of the building described in the paper they were faced originally with those high and low bays; but Mr. Hargreaves was very helpful and at Mr. Bullen's request the bays had been altered to be the same height. He believed that was really the answer !

There was no provision for servicing cranes, apart from providing access gangways at all points. As to the provision of more joists, presumably somewhere up in the roof, the low-bay cranes could be run out to the high bays where the high bay cranes could service them ; but he was not quite sure about the high bays.

After indicating that he would try to answer Mr. Warburton's more technical points in writing, he said Mr. Lewis always asked difficult questions. The 3 in. square billets were of mild steel and there was nothing special about them ; they were welded intermittently at 18 in. centres. They were not allowed for in the moment of inertia of the beams; they might not be there at some time. However, he was very glad to hear that Mr. Lewis considered welding to the girders to be the best practice, and he agreed.

Dealing with the stiffeners to the crane girders, he said the main stiffeners were at about 4 ft. 6 in. centres ; but towards the centre of the span it was thought that the compression flanges were getting a little slight, and small triangular stiffeners had been introduced between the main stiffeners.

He agreed very much with Mr. Lewis' remarks concerning the mis-alignment of crane tracks; and he would not be surprised if we needed to give a little more attention to the design of cranes rather than to the design of buildings. The point was a good one, but perhaps he had better not say more about it.

He did not remember exactly the method of jointing the billets, but he believed they were just butted on the tops of the girders at their ends. The billets were kept short at the ends by 1 ft. or 9 in., and then a short piece was put in between the ends of the two billets, overlapping or running over the joints in the girders. The short pieces could be taken out and replaced if necessary. They were welded to the girders throughout.

Thanking Mr. Palmer for his comments, Mr. Bullen said he liked very much broad flanged beams, and he would like to see as many of them and as many varieties of them as possible readily available. Continuing, he said that certainly he had designed the holding-down bolts, but had not specified any particular method of fixing and installing them ; the method adopted was due entirely to the contractors, and it had worked extraordinarily well.

There was some play in the holes before tightening up and grouting ; otherwise, the templates were exact replicas of the column bases.

Mr. Bullen agreed with Mr. Palmer that the " pepper-box " form of roof lighting was not necessary in a tall building.

The foundations were on piles. We did not know a great deal about piles, but perhaps we knew a little more about them than about the earth into which the piles

were driven. However, there had been no troubles so far.

As to the "carry-over" and the number of pins, each pin consisted of a piece of round rod which was laid in between two semi-circular grooves in the faces of the plates. The clearance between the faces was filled in with mastic at ground level; up above this was not thought necessary. As far as he could tell, there was flexing; the movement experienced when the roof sheeting was put on had indicated that that was so.

Referring to his model to illustrate an important feature of the structure, Mr. Bullen pointed to the overhanging arms in the middle bays, and said that if the centre row of stanchions were taken down, the building would remain stable; but the cranes could not run through the two middle bays, of course!! He suggested that anyone who wanted to build a garage, a warehouse, an arena, a cinema, or similar building, could use that form of construction, which gave about 175 ft. span.

(Mr. Bullen had the centre row of stanchions in the model taken down, to illustrate the flexibility of the design.)

The PRESIDENT thanked Mr. Bullen for his handling of the discussion.

Written Contribution

Mr. R. P. KEY writes: I would like to support the view expressed by Mr. Bullen during the discussion that we should not forget that engineering is an art as well as a science. The purely theoretical approach can lead to over-emphasis on the less important features of structures and standardisation can retard future progress by establishing practices so rigid that future progress is made more difficult. This outlook on progress is exemplified by the sequence of thought which led to the overall layout of the new factory.

To supply the structural steel designer, the architect, and the sub-contractors with the layout shape, the main dimensions, the crane loadings, etc. of the proposed factory, some twelve months' investigation work was required.

The terms of reference could be only of a vague nature, and included directions that the company's products would become larger, heavier, of better quality, more complicated and of increasing variety. In addition to the future developments of the products, it was considered that the rate of obsolescence of process methods and of process machinery would be higher than in the past, but just on what and by how much could not be answered completely by the purely scientific approach. It was possible, of course, to define a fairly accurate production programme for the next two or three years, and to lay out on paper, and to plan with models, operation sequences, work-in-progress areas, and so on. Associated with these estimates some 30 to 40 schemes and layout arrangements were drawn, and considered, but all through this work questions arose about the position ten, fifteen and twenty years hence.

Finally, it was decided to have a layout which would allow the movement of heavy engineering products, when in the sub-assembled state, from any point in the factory to any other point, by overhead cranes. Many questions, such as the number of cranes in each bay, were answered with this object in view. Also, it was decided to arrange that the flow of work and process machinery could be changed with a minimum of cost, and without affecting the material movement facilities. These decisions, together with the need of a factory plan which could be expanded in a logical way led to the present layout.

In comparing Mr. Bullen's steelwork design with that of other factories, or with other designs, it is important to bear in mind that the main steelwork should be viewed as part of a large machine designed primarily for material movement and with a number of special features, including provision for future extensions in a pre-arranged manner.

Mr. Bullen has provided a design which should require low maintenance, and it is clear that he has recognised that the cost of sub-assembly fabrication and erection should be considered along with steel material savings.

Written Reply by the Author to Sundry Questions

There is no doubt but that the unusual character of this design has given rise to plenty of questions. I am glad to notice that these questions have been directed towards the economy of structural design and it is particularly gratifying to find that the design of the foundations is regarded by other engineers also as part of the design of the whole structure. On more than one occasion I have made the point that it is impossible to design economically unless the whole structure is taken into account. Several speakers at the discussion raised this question, and I think it would be a fair statement that the 10 per cent. saving in steel weight achieved on the transverse bays was probably offset by a slight increase in weight of steel for the foundations, but this increase would be no more than 2½ per cent. The saving in cost was much greater than 10 per cent. and there is no doubt that even allowing for a slight increase in the cost of the foundations, the final saving in cost was still much more than 10 per cent.

I am grateful for the suggestion made by Mr. Husband that the roof members might have been treated as portal frames from crane rail level upwards, but I do not know whether this would have made it possible to use rolled sections throughout; I rather think this would not have been so as some appreciable sections would have been required at all the gutter levels.

Mr. Warburton's suggestion that the stanchions might have been placed under the crane beams on the external columns is a very good suggestion; it would have involved perhaps a somewhat unusual arrangement at the top section of the column, but, as he said, might have led to a further improvement in the general design. With such an arrangement, of course, the columns would have encroached more upon the floor space, but this would not have mattered very much. On the other hand, had some similar arrangement been adopted for the internal columns, the more normal braced composite columns would have resulted and the clean arrangement and lines of the building as designed would not have been achieved.

Regarding the general proposition put forward by several speakers, namely, that the total strain energy of the structure must be equal to the external work done, whilst I cannot disagree with that as a theoretical proposition, I do say again that economy comes more by a careful arrangement of members than by finesse in detail design. Futhermore, economy also comes by the use of members readily obtainable such as rolled steel sections, and any arrangement of a structure which makes use of rolled sections will probably result in ultimate economy. It would be fair to say that if rigidity had been utilised at all the joints then difficult rigid connections would have been necessary; as Mr. Warburton rightly says, Dr. Hendry has been at great pains to show that haunches provide dividends out of all proportion to their cost, but it must not be overlooked that haunches at the tops of the columns would have made it necessary to raise the roofs to obtain the necessary crane clearances.

Reverting to the questions of foundations, the conditions at this site were such that piles were necessary; consequently, the addition of a few extra piles at appropriate points to withstand moments involved relatively little additional cost. On the other hand, had the ground been such as to make piling unnecessary, then there might well have been sound arguments in favour of making the superstructure continuous but supported on pins, since it is very difficult to achieve a reliable degree of fixity in columns founded upon yielding ground. Thus, once again, it may be emphasised that the design of the superstructure is closely bound up with the design of the foundations. The sliding joints referred to by Mr. Husband were so arranged as to cause the 16 × 6 joist rafters to slide upwards as the built-up welded rafters deflected; thus, at the two internal ridges flexibility of structure was achieved, and the continuous rafters were free to deflect without introducing serious movements on the remaining rafters.

In reply to Mr. Lewis's questions about the crane rails, these were normal 3 × 3 billets of mild steel and they had no "ears" by means of which they could be welded to the flanges. The intensity of crane wheel pressure could be worked out from the fact that the reaction on the wheels was taken as 20 tons exclusive of impact, the diameter of the wheels was 24 inches and the width of the wheels was 3 in., the wheels having double flanges.

I am very grateful to Mr. Bondy for his remarks, but I think I have dealt already with his reference to continuous structures. I fully appreciate Mr. Bondy's and other speakers' remarks but we require not only to achieve economy of steel but also economy of cost, and it is questionable whether a fully continuous structure is always as economical as a discontinuous one.

The top flanges of the large crane girders were $2\frac{1}{4}$ in. thick throughout.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, June 26th, 1952, at 5.55 p.m. Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership?

Students

Allman, Laurence Mills, of Johannesburg, South Africa.
Balaam, David John, of Harrow, Middlesex.
Barlow, Eric, of Stockport, Cheshire.
Bateman, Douglas James, of Chessington, Surrey.
Butler, Alan, of Manchester.
Cox, Bruce Albert, of London.
Cumberlidge, Philip, of Macclesfield.
De Penning, Jean Charles, of London.
Edwards, John William, of Stanmore, Middlesex.
Fisk, Roy Terence, of London.
Folly, Eric John, of Johannesburg, South Africa.
Fyall, Gordon Bryan, of Johannesburg, South Africa.
Geddes, Brian Sydney, of Manchester.
Girling, John Marcus, of Ashford, Middlesex.
Hewitt, Eric, of Warrington, Lancs.
Hopkin, Elwyn Edward, of Bridgend, Glam.
Hughes, Glyn, of Hereford.
Hunt, Henry William, of Thornton Heath, Surrey.
Jackson, Norman, of Sheffield.
Kenny, Alphonsus Jerome, of Wallasey, Cheshire.
McKenzie, Alexander Stewart, of Salford, Lancs.
Milligan, William Ramsay, of Manchester.
Munsch, Anthony Mark, of Brighton.
Noronha, Alan Demonte, of Mombasa, Kenya.
Nuttall, Peter, of Southall, Middlesex.
Ogilvie, David George, of Stockport, Cheshire.
Parry, Robert Tudor, of Eastham, Cheshire.
Reader, David, of Ilford, Essex.
Rutherford, Michael William, of Mombasa, Kenya.
Smith, Roy, of Todmorden, Lancs.
Thompson, Gordon, of Manchester.
Tietz, Stefan Berthold, of London.
Wajdowicz, Pawel, of London.
Wasdell, Edwin John, of Sutton Coldfield.

Graduates

Axtell, Roger John, of Ascot, Berks.
Baguley, Maurice Grant, of Bexleyheath, Kent.
Balsara, Nāriman Shapurji, of Bombay, India.
Baxter, Roy Antony Hilder, of Worthing.
Bird, Brian Cecil, of Ambergate, Derbyshire.
Bowman, Alexander Hughes, of Johannesburg, South Africa.
Collingwood, Geoffrey Frank, of Leeds.
Dagger, Graham Dennis, of Edgworth, Lancs.
Danks, Arthur Edwin Henry, of Tipton, Staffs.
Faulkner, William Lilwall, of Liverpool.
Fleischer, Kurt Karol, of Cheadle Hulme, Cheshire.
Gauld, Forbes Fraser, of Manchester.
Haines, Peter James Terry, of Birmingham.
Halsall, John Denys, of Salford, Lancs.
Hardingham, Geoffrey, of Ilkeston, Derbyshire.
Harrison, Arthur, of Maghull, Lancs.
Hill, John Worsley, of Ramsbottom, Lancs.
Hinder, Stanley James, of Bristol.
Hodgkinson, Arthur, of Moston, Manchester.
Ironmonger, Albert Edward, of Rainham, Essex.
Jezierski, Andrzej, of London.
Jones, Kenneth William, of Treorchy, Glam.
Kay, Marzell, of Leamington Spa.
Kharote, Narahar Vishnu, of Poona, India.
Kirkpatrick, Peter Robert Johnston, of Newcastle-upon-Tyne.
Langdon, William Albert John, of Gloucester.
Longley, Geoffrey Wray, of Johannesburg, South Africa.
Mahalingam, Selvadurai, of Colombo, Ceylon.
Mann, Ronald Henry, of Nottingham.
Millar, William Donald, of Risley, Lancs.
Muir, Ian Stanley, of Newcastle-upon-Tyne.
Munnings, John Arthur, of Enfield, Middlesex.
Murray, Robert Carr, of Billingham, Co. Durham.
Naylor, Gerald Wilfrid, of Harpenden, Herts.
Newsome, Arthur Sheard, of Whitley Bay, Northumberland.
Platt, Derek William, of Purbrook, Hants.
Powell, Richard Thomas, of Cardiff.
Richardson, Ronald Edward, of London.
Riddick, Alan Moir, of London.
Rowe, Peter Walter, of Manchester.
Sheerman-Chase, Denis, of Morden, Surrey.

SMITH, Reginald George, of Edinburgh.
SRINIVASAN, Kannurpatti Venkataramanan, of Bombay, India.
SUMMERSCALES, Walter Keith, of Halifax.
TINSLEY, Peter Hugh, of Risley, Lancs.
WOOLNOUGH, Edwin Arthur, of Manchester.

MEMBER

DIXON, Benjamin Joseph, of Dublin.

TRANSFERS

Students to Graduates

CORSER, Thomas Herbert, of Manchester.
JARVIS, Anthony Peter, of London.
LEACH, Alan Brickett, of Watford, Herts.
MCGREGOR, David Hugh, of Manchester.
ORRELL, Harold, of Bolton, Lancs.
POZZO, Lucian Mario Paolo, of London.
SMITH, Peter Frederick, of Manchester.
WALMSLEY, Joseph Roy, of Bolton, Lancs.
WILMAN, David Richard, of Belvedere, Kent.

Graduates to Associate-Members

BONE, Arthur, of Middlesbrough.
BOWMAN, Arnold, of Billingham, Co. Durham.
BUNCE, John Wallace, of London.
SUNDLO, Harald, of Karachi, Pakistan.
TELLER, Otto George, of London.

Associate Members to Members

COSWAY, Donald Henry, of Orpington, Kent.
DESHPANDE, Chintaman Vishnu, of Bombay, India.
FAREBROTHER, James Edward Charles, of Ewell, Surrey.
HENDERSON, John William, of Hemel Hempstead.
HOUGH, Norman Edward, of London.
JONES, Owen William, of Derby.
MASON, Alexander, of Edinburgh.
PEATFIELD, Alfred Edgar, of London.
WALLACE, Charles Ross, of Edinburgh.

Member to Retired Member

MILTON-COLE, Frederick, of Bulawayo.

RE-ADMISSIONS

Associate-Membership

CROUCH, Reginald Henry, of Kenton, Middlesex.
EPHRAUMS, Frederick Thomas, of Singapore.
MILNE, James Young, of Newbury, Berks.

OBITUARY

The Council regret to announce the deaths of Leslie St. Clare RUNDLETT (Member), Walter Albert SCOTT (Retired Member), and Derrick Dudley ROSE (Associate-Member).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of William Henry JOHNSON (Member) and John SHARDLO (Graduate).

EXAMINATIONS—JANUARY, 1953

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on January 6th and 7th, 1953 (Graduateship) and January 8th and 9th (Associate-Membership).

REPRESENTATION

The Council have re-appointed the following members to be the Institution's representatives on the Joint Committee on Materials and their Testing :—

Mr. W. H. Woodcock (Hon. Editor).

DEPUTY

Mr. Ewart S. Andrews (Past-President).

PERSONAL

Professor W. T. Marshall (Member) has been appointed Regius Professor of Civil Engineering in the University of Glasgow.

* * *

Mr. J. L. Wheeler (Member) announces that he has taken his Senior Assistant, Mr. Arthur H. Jupp, D.F.C. (Associate-Member) into partnership. The business will be carried on under the title of J. L. Wheeler & Jupp at Empire House, St. Martins-le-Grand, E.C.1.

* * *

Mr. W. A. Mitchell (Member) has set up in practice as a Consulting Engineer at 81, Chester Square, London, S.W.1.

* * *

Mr. J. R. Hill (Associate-Member) has retired from the Ministry of Transport and has taken up an appointment with the Air Ministry as Section Officer, Air Ministry Directorate-General of Works, R.A.F. Station, Andover, Hants.

* * *

Mr. A. E. Peatfield (Associate-Member) has been appointed Standards Engineer to the Anglo-Iranian Oil Company, Ltd.

RESEARCH AWARDS

The Council have instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :—

(a) investigations of an experimental or analytical character ;

(b) studies of historical or statistical records ;

(c) improvements in principles or methods of construction ;

(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;

(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms :— A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors

must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :—

(a) the nature of the subject and its conclusions ;

(b) the value of the paper in advancing the science and art of structural engineering ;

(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1951, and September, 1952, is October 31st, 1952.

COMMUNICATIONS TO MEMBERS

The Institution is experiencing difficulty in receiving replies to communications addressed to the undernoted members, and the Secretary would be glad to be notified of their present addresses as soon as possible :—

A. Cole (Member).
W. S. Blount, G. A. Dewar (Associate-Members).
D. J. Close, Abdur-Rahim Khan, S. Panchanathan (Graduates).

ADDITIONS TO THE LIBRARY

The following volumes have been presented by Major C. G. J. Lynam :—

The Modern Bricklayer, Vols. I, II and III. London (undated).
The Modern Practical Plumber, Vols. I, II and III. London (undated).
The Modern Plumber and Sanitary Engineer, Vols. 1, 3, 4, 5 and 6. London, 1912.
Standard Methods for the Testing and Analysis of Cements. G. & T. Earle, Ltd., 1901.
American Concrete Institute Proceedings, 1922, 1924, 1925, 1926, 1927, 1928 and 1929.
Canadian Engineer, 1922-23.
Concrete, 1917-23 and 1925.
Concrete and Constructional Engineering, 1917, 1918 and 1919.
Electrical Review, 1919.
Engineering and Contracting, 1918-19 and 1922-24.
Engineering News Record, 1917-19, 1922 and 1923.
General Electric Review, 1919.
Genie Civil, 1918-19.
Municipal Engineering, 1917-19.
Surveyor, 1913-14.

ANNUAL DINNER

The Annual Dinner of the Institution will be held on Thursday, October 2nd, 1952, at the Dorchester Hotel, London, W.1, at 7 o'clock for 7.30 p.m., when the Principal Guest will be the Rt. Hon. Lord Woolton, C.H., Lord President of the Council.

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting will take place on Saturday, August 23rd, when a visit has been arranged to the Coryton Oil Refinery. A coach will leave 11, Upper Belgrave Street, S.W.1, at 9 a.m., and will return from Coryton at 12 noon, arriving in London about 1 p.m. The fare will be about five shillings.

A visit is being arranged to the Steel Mills of Messrs. Dorman Long & Co., Ltd., at Middlesbrough. The party will leave London on Thursday night, October 9th, visit the mills on the 10th and return on Saturday, October 11th. All meals and accommodation will be arranged, but the names of those wishing to attend must be sent in at once as the number of seats available is limited.

Hon. Secretary : C. Allen Browne, 43, Coolgardie Avenue, Highams Park, London, E.4.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 28, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

Hon. Secretary : E. R. Deeley, A.M.I.Struct.E., Arranmoor, Adshead Road, Dudley, Worcs.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : M. H. Evans, B.Sc., 107A, Metchley Lane, Harborne, Birmingham, 17.

NORTHERN COUNTIES BRANCH

Hon. Secretary : Ian MacGregor, M.I.Struct.E., Messrs. H. Pickup, Ltd., Roscoe Street, Scarborough.

NORTHERN IRELAND BRANCH

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., "Lisleen," 13 Finaghy Road North, Belfast, Northern Ireland.

SCOTTISH BRANCH

Hon. Secretary : D. G. Drummond, B.Sc., M.I. Struct.E., A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. Harding & Sons, Ltd., 10-12, Market Street, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

Hon. Secretary : C. E. Saunders, M.I.Struct.E., Dunkery, Edward Road, Walton St. Mary, Clevedo, Somerset.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

The Johannesburg section of the Branch had a most enjoyable evening on April 1st, when Mr. L. E. Scott White (Past-President) gave a talk on the Re-siting of an Historical Building in Whitehall.

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

Modern Trends in Steel Construction*

By Professor Dr. F. Stüssi (President of the International Association for Bridge and Structural Engineering)

1. The evolution of steel construction, like any evolution, displays external and internal characteristics. The external characteristics are nearly always more evident, and can thus be more easily recognised than the internal ones. On the other hand, it is very often only the internal or intellectual characteristics that make evolution possible. In steel construction particularly, the external evolution and the great progress made during the last 100 years would not have been possible without the evolution of the corresponding scientific bases.

On the following pages, I shall try to show some of the characteristic trends of modern steel construction. I propose to start with the consideration of some external characteristics, and we shall very soon see that the external characteristics are in very close connection with theoretical reasonings, which are most important for the progress of the art.

2. Among the external characteristics that are most evident when making a comparison between important steel constructions built at different times, mention should be made in the first place of the increase of the attainable span length, the simplification of structural forms, and the introduction of a new connecting means, i.e. the welding technique ; these are in my opinion the most important ones.

The limit of 1,000 m, or about 3,300 ft., for a single span has been exceeded by a steel bridge of our times. Such span lengths thus form a characteristic of modern bridge building. I would here confine myself to a short comparison of the most important suspension bridges which have been erected (Fig. 1). The conception of theoretical weight, given by the author in his paper at the 1948 Congress of the International Association for Bridge and Structural Engineering,[1] shows the connection between the load and the dead weight of a structure in a very simple form for any given system and any given material. For each system and for each material there is a certain limiting span which cannot be exceeded. Starting from this limiting span, we can easily derive the notion of an economically advisable span, for which the amount of material involved is at the limit of sound relation to the live load. For suspension bridges and other material which is used to-day for this system (high-tensile steel wire), we can show that the suspension bridges built up to now have not yet reached this economical span limit, and it will be possible, economically and without exceeding technical difficulties, to build suspension bridges with still greater spans. It has been confirmed in the meantime by the design of a suspension bridge of more than 1,500 m or 5,000 ft. span, proposed by Dr. Steinman for a bridge across the Straits of Messina. We can thus see that theoretical reasoning about the limiting or economical span length of a certain system is not only interesting from a theoretical point of view, but also directly useful in practice.

Besides the suspension bridge type, the arch system and the cantilever system, both in the form of trusses, are adopted for long span steel bridges. For arch bridges an increase of the span length within economically advisable limits still seems possible, whilst the economical limit for the cantilever system has, in my opinion, already been reached or even passed. It seems to be a research problem of the near future to find out whether or not the economical span limit of the cantilever system, using the structural steels known to-day, can be increased by a suitable arrangement of the system and a refined design of the structural details.

A second external characteristic in the evolution of steel construction is to be seen in the simplification of the structural forms. This characteristic trend is clearly recognised from a few simple comparisons. If we compare, for example, an arch bridge built about 50 years ago (Fig. 2) with a corresponding present day structure (Fig. 3), or if we compare the interior view of one of the great cantilever bridges such as the Quebec Bridge (Fig. 4) with the towers of one of the new suspension bridges (Fig. 5), we can easily see the tendency towards simplification, in the arrangement of the structure as a whole and in the individual structural details. The tendency to use full web members instead of lattice bars within economical limits can only be realised in connection with a refinement of our design methods. For the application of members composed of thin plates, it is necessary to increase our knowledge of the problems of elastic and plastic stability. This tendency to prefer full web members does not, however, mean the elimination of trusses ; the truss system will remain in favour for long spans in the future, but for this system too, the demand for simple and clear structural forms in the arrangement of the system and in the structural details is of prime importance. We can see from the example of an existing truss bridge (Fig. 6), that this type can offer technically and æsthetically good solutions.

The possibility of simplifying the elements of steel construction is greatly supported by the welding technique introduced about 20 years ago in addition to the classical means of riveting and bolting. It seems to me quite clear that the possibilities of the welding technique will only be fully utilised after profound and systematic research work in the future. I do not intend to discuss this aspect of the welding problem. I will limit myself to mentioning the excellent paper by Professor F. Campus in the Preliminary Papers of the Building Research Congress, London, 1951; I entirely agree with this paper. But I would like to mention some structural problems that are important for the structural-steel engineer.

Regarding the design of the assemblage point of a truss girder, it is not admissible simply to transfer the customary structural forms of a riveted truss to the welded system. We have to realise the fundamental structural requirements of the welding technique, namely to avoid abrupt changes of the sections and the accumulation of welds, and we must therefore find an

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* *Lecture delivered on the 14th September, 1951, before the British Group of the International Association for Bridge and Structural Engineering at the Institution of Structural Engineers, 11 Upper Belgrave Street, London, S.W.1.*

[1] *F. Stüssi : "Theoretical weight as the basis for selecting a type of bridge." International Association for Bridge and Structural Engineering, Third Congress, Liege, 1948. Final Report, p.475 et seq.*

appropriate form of section of the truss members. This requirement leads to the arrangement of a welded assemblage point as shown by way of example in Fig. 7 and to the adoption of curved flanges. The inconvenience of such curved flanges, namely the unequal distribution of stresses across the flange width due to the radical forces, and consequently, the poor utilization of the material, will be more and more reduced as the radius of curvature of the flanges increases. This leads to the conclusion that, for welded trusses, the number of members to be connected at one assemblage point should be reduced as far as possible. Therefore, the strut bracing or finally the Vierendeel-girder are appropriate stances arising most frequently in practice, leads to a very simple calculation.

In this connection I have further to mention the simplifications due to welding in the field of the steel frame building. The design of the connection between the beams and stanchions is possible in different forms ; Fig. 8 shows an example.

3. As a result of the tendency towards full utilization of the material in every part of a structure, the box section has been developed and applied. I shall here mention only two typical examples : in bridge as well as in sluice construction, box girders have been executed or designed, utilizing the roadway slab or the planking,

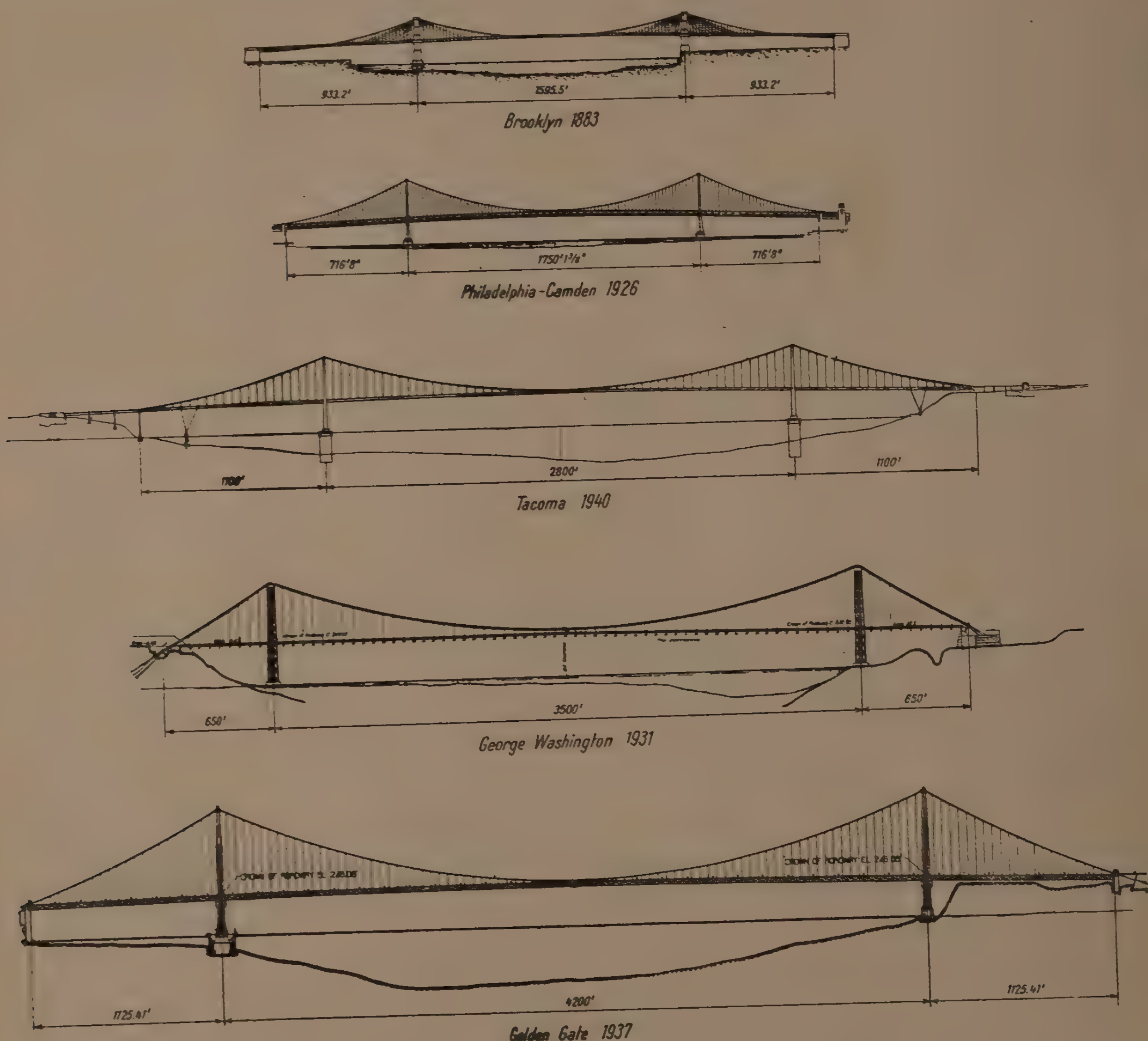


Fig. 1

systems for welding. Hitherto the Vierendeel-girder has been used mainly in Belgium, but I think that its adoption in other countries will only be the logical consequence of the evolution of welding technique. For the successful application of the Vierendeel-girder it is necessary to have efficient and exact methods of calculation ; without such a basis, the application of a Vierendeel-girder would be a drawback for the whole development. In Volume 10 of the " Publications " of the I.A.B.S.E. I have given the outlines of such a method, which can be applied in the most general case without difficulty and which, under the special circum-

together with the main girders and the bracing, as closed element. The form of the box girder is not new and in particular British engineers can be proud of the Britannia Bridge built by Robert Stephenson, who successfully realised this form of construction on a large scale more than a century ago. With experimental means, he had to solve, for the execution of this bridge, a series of stability problems, which even to-day are not completely solved for the general case. The experiments carried out by Stephenson and his staff led him step by step to the well-known form of his Britannia Bridge. For such a box girder we have not only

consider the stability problems on an experimental basis, but we also need an exact method of stress computation.

This problem, stress computation in a box girder, has been solved only for a few simple cases applied in aircraft construction. In our domain, however, we have to deal with more general cases, corresponding to the different possible forms of our bridge or sluice sections.

Fig. 2

This problem, which may be characterised by the notion of the centre of shear or the centre of torsion, shows the peculiarity that in the case of closed or box sections the shear stresses can no longer be computed from equilibrium equations, but we have to deal with a statically indeterminate problem for determining the shear stress at a certain point of the section. For symmetrical open sections the classical theory of Navier, based on the assumption that plane sections remain plane after

Fig. 3

deformation, leads, for the normal applications in steel construction, to a stress computation sufficiently accurate for the design. For box sections of any form, an analogous method with the same degree of accuracy can be derived, on the assumption that the form of the section remains unchanged during deformation. The outlines of this method, which can be applied without difficulty and with a reasonable expenditure of time, will be published in the next volume (11) of the "Publications" of the I.A.B.S.E.

4. For the choice of a system, different normal types are at our disposal. If we limit ourselves to constructions of one single span we have the beam, arch and suspension systems. The comparison shown in Fig. 9 demonstrates that those normal systems are closely related to each other. From a truss beam with polygonal upper chord, we can derive the arch by eliminating the lower chord and by assigning its functions to the abutments ; the web-members, which are stressed only in the case of unequal distribution of the loading, are to be replaced by executing the upper chord as a stiff full web member.

In steel construction, in bridge as well as in building construction, we have sometimes to deal with problems where the normal systems treated in the text books do not give the best solution, but where we have to find, by

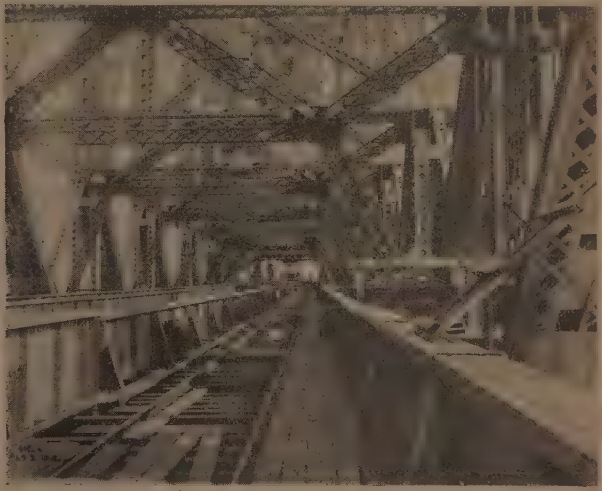


Fig. 4

theoretical reasoning, special arrangements to fulfil the requirements of the particular case. I would mention as an example the steel construction for the new hangar of the airport at Kloten (Fig. 10). There the building authorities laid down that in view of fire risks, the steel construction should be supported by a concrete structure

Fig. 5

at least 6 m above the floor. The external form of the roof was also prescribed. Owing to unsatisfactory soil conditions, the horizontal pressure of the roof construction had to be reduced as far as possible. Starting from the prescribed form of the roof, two normal solu-

tions could be taken into consideration, but the first, a simply supported beam, would have led to an excessive structural weight, and the second, the arch, would have given too great horizontal pressures. We therefore proposed an asymmetrical three-hinged arch, which avoided these two disadvantages and allowed an economical execution. The solution adopted (Fig. 11) required a steel weight of 18 kg/m² for the roof truss.

Fig. 6

5. Starting from external and externally visible characteristics of the evolution of steel construction, we were also able to determine some internal characteristics ; the total evolution of steel contruction is imaginable only on a scientific basis and in close connection with the evolution of structural statics. In no other branch of structural engineering are design and theory so closely related to each other, and the evolution of theory forms an essential part of the development and the application as a whole.

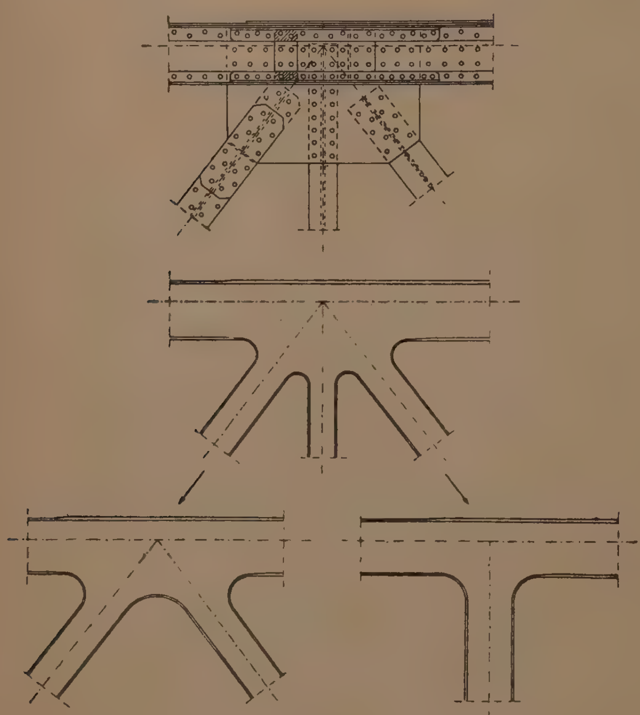


Fig. 7

The evolution of characteristics are not always apparent in the external form of a construction, and there is to-day a very interesting discussion in our profession about such a problem, which is not externally apparent but influences the design and the methods of design in principle. This is the problem of plasticity which was also discussed on a large scale at the Building Research Congress, London, 1951. I should like to limit myself to mentioning only two items of this problem, namely the plastic buckling of compressed bars, and the plastic or limit design.

In the buckling problem it is very interesting to find that as early as 1826 the great French engineer, Louis Navier, in his fundamental book " Résumé des lecons sur l'application de la mécanique " gave some single values for the plastic buckling stresses for timber and wrought iron, the two most important building materials used in those days. Navier's knowledge about buckling evidently did not hold the attention of the profession and was lost later on ; it is a fact that only 70 years afterwards, Ludwig von Termajer gave an empirical formula and Friedrich Engesser a theoretical solution for inelastic buckling. The empirical formula of Tetmajer is very simple and easy to handle, and is therefore still used to-day for design in many countries. There is no need to deal with this formula here. I would like to state, however, that Engesser corrected his first proposition by taking into account an objection of Jasinski, thus introducing the buckling modulus which we know as the Engesser modulus, or since 1908 as the Engesser-Karman modulus. Only 50 years later Shanley pointed out that the first proposition of Engesser was the correct solution of the plastic buckling problem, and I therefore think we should call this theory the Engesser-Shanley theory. This short retrospect on the history of the plastic buckling problem shows a fact which to our regret can be seen time and again in engineering science, namely a certain inertia of the profession, as Professor Winter calls it in his paper for the Building Research Congress, 1951. Besides that, there is another reason why the inexactitudes of the Engesser-Karman theory have not been discovered for such a long time : for a steel with a distinctly marked yield limit which is only a little above the proportional limit, the two theories Engesser-Karman and Engesser-Shanley give approximately the same

Fig. 8

results ; the difference between the two theories being apparent only for a steel with a pronounced transition zone between the proportional and the yield limit. Fig. 12 demonstrates this fact. I have no doubt that future specifications for steel construction will be based

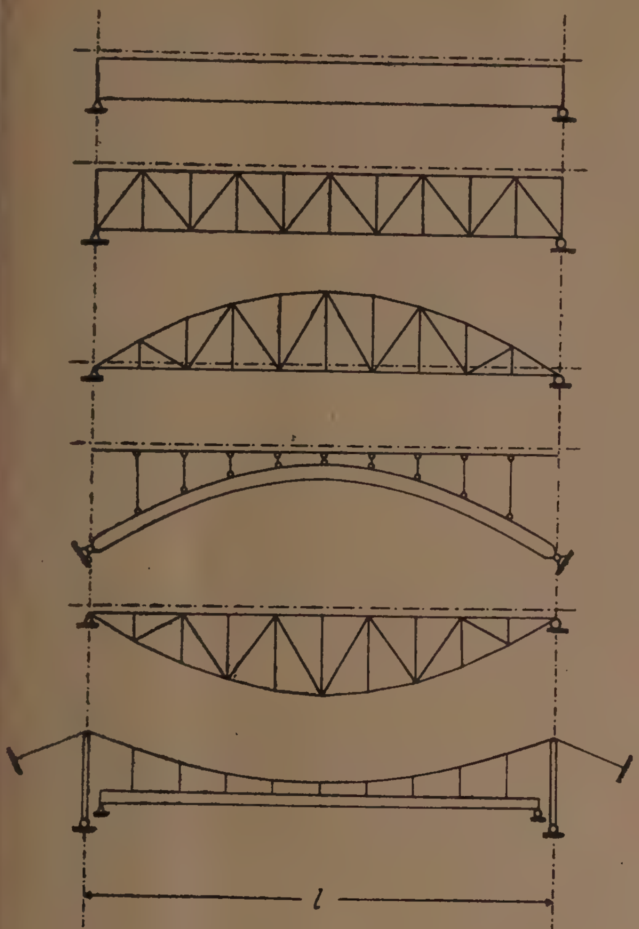


Fig. 9

on the Engesser-Shanley theory, the results of which are on the safe side.

The English-speaking section of the profession recently seem to be very interested in the problem of limit or plastic design, just as the German-speaking section was about 15 or 20 years ago. I should like to point to some theoretical considerations and corresponding experiments we carried out and published in 1935.[2] I then desired to find out if the factor of safety for continuous beams designed according to the plastic design or method of plastic hinges was the same as for a normally designed corresponding simple beam. For the solution of this problem it seemed to me to be important to realize a case of loading for which the question could be answered direct by experiment. Following a theoretical reasoning, we tested a continuous beam with three spans by a single concentrated load in the middle of the midspan (Fig. 13). If the plastic hinge method was correct, this beam should carry double the load of the simple beam, independent of the span length of the side spans. This, however, is not

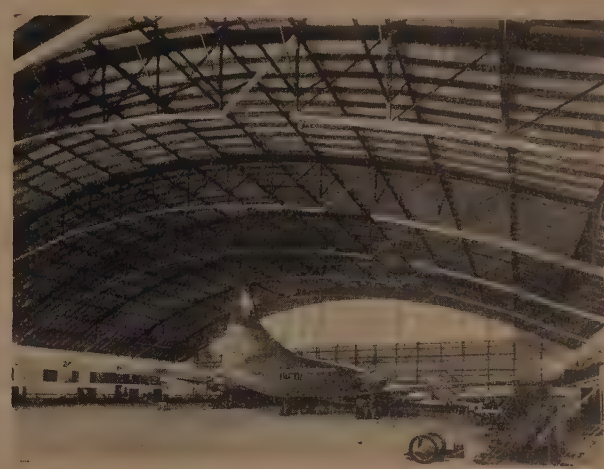


Fig. 10

possible according to theoretical reasoning, because we cannot imagine a sudden transition from the behaviour of the continuous beam to that of the simple beam. The experiments confirmed this reasoning as clearly as could be desired. The discrepancy between the actual behaviour of the continuous beam and the plastic hinge method can be explained by the fact that, after the proportional limit has been passed, the moments in the middle of the midspan and at the supports show a certain tendency towards equalisation ; but this equalisation is not complete, and with increasing load there is again a divergency (Fig. 14). As the plastic hinge method supposes complete equalisation of the moments, this method is not correct, and even for a purely statical loading this method does not ensure the required factor of safety. The results of the elastic theory, on the other hand, are on the safe side. I am convinced, therefore, that the application of the limit design would reduce the factor of safety of our constructions and it is necessary to retain for statically indeterminate systems the design

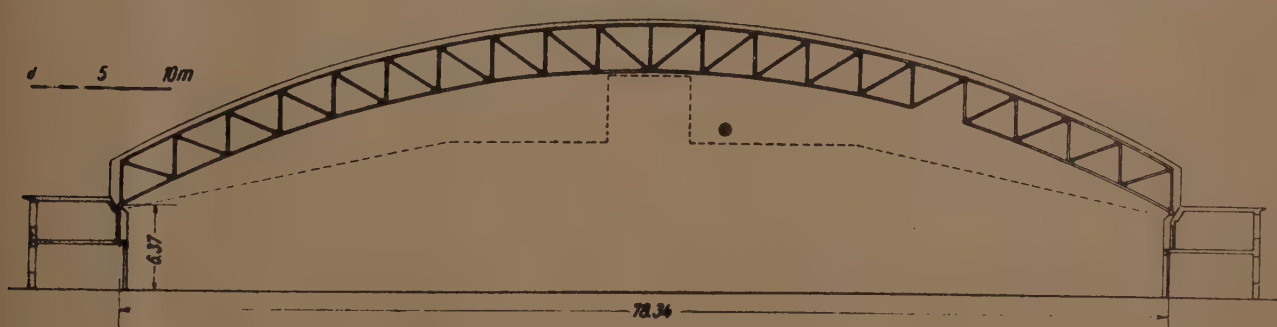


Fig. 11

[2] F. Stüssi and C. F. Kollbrunner : *Beitrag zum Traglastverfahren. "Die Bautechnik"* 1935, *Heft* 12.

based on the elastic theory, not only for dynamic but also for static loading.

6. The evolution of steel construction is not yet finished but is still going on. Its external characteristics are dominated by a marked tendency towards a simplification of the structural forms in the total arrangement of the system as well as in the design of the details. This simplification consists of omitting all accidental and unnecessary parts.

In this connection we can understand why full-web beams are more frequently applied than in former times; but this tendency has its natural limits set by technical

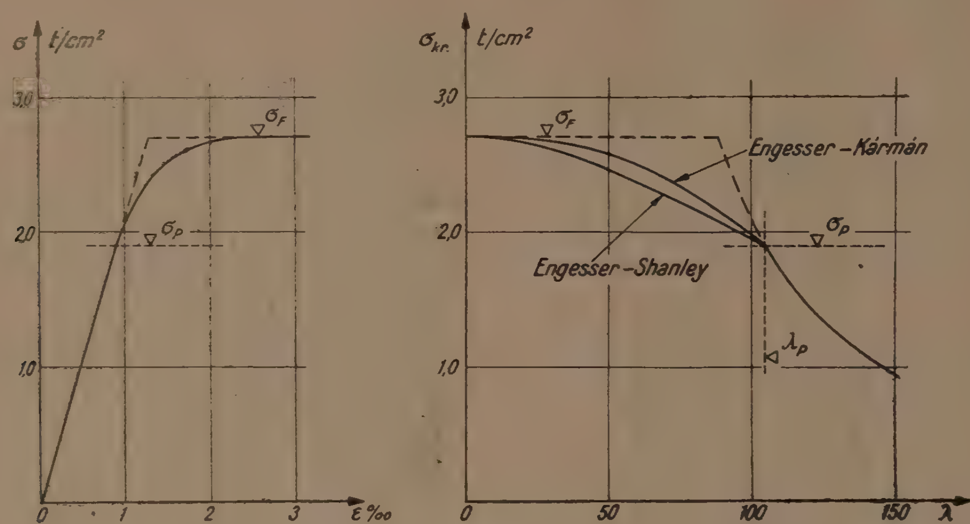


Fig. 12

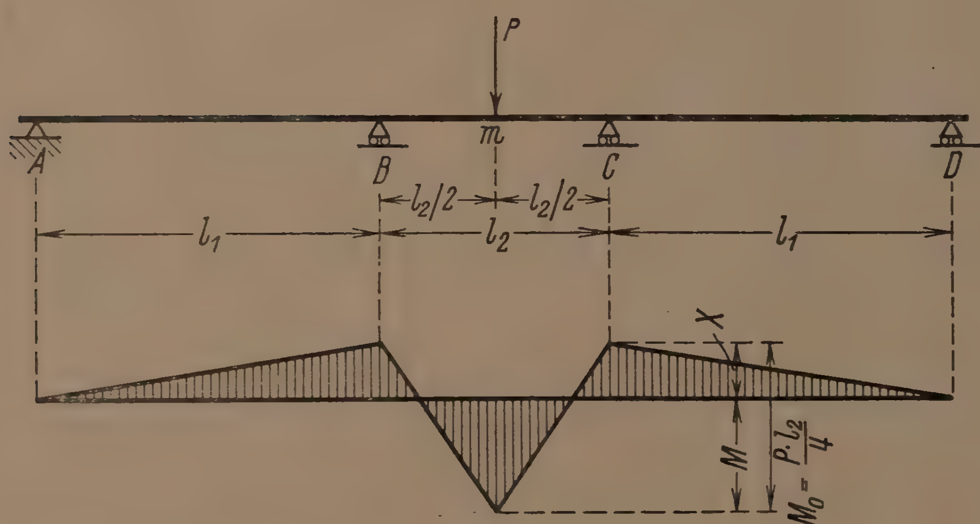


Fig. 13

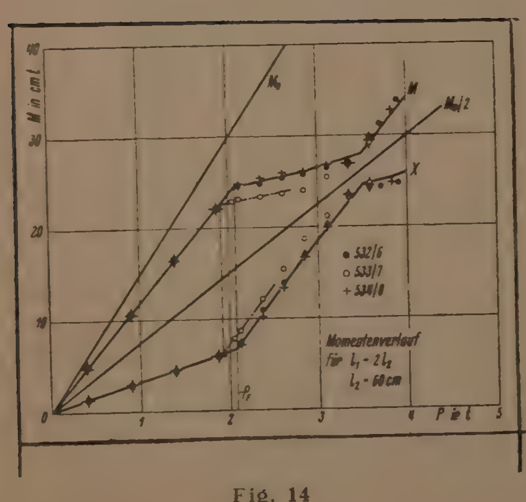


Fig. 14

possibilities and economical reasons, and I am sure that the truss system will still be used in the future for long span structures. For medium spans, I think that also the Vierendeel-girder in connection with the evolution of welding technique will be more often adopted than is the case at present. But also for the welding technique itself, we have for the moment to state limits of application, and the classical means of connection, rivet and bolt, will also play their part in the future.

The evolution of the theory is not always continuous, as we can see from the example of the buckling problem. Despite the advantage of teamwork in research, the intellectual evolution has to be supported by the ideas and reasonings of prominent individual scientists.

Representative steel construction is not only a technique or technical science; design and execution must be based not only on experience and theory, but also on artistic and scientific intuition.

Accuracy of Determination of the Elastic Torsional Properties of Non-Circular Sections Using Relaxation Methods and the Membrane Analogy

By W. B. Dobie, M.Sc., Ph.D. and A. R. Gent, B.Sc., Ph.D.

Summary

The paper describes an investigation into the accuracy of determination of the torsional properties of a non-circular section by the membrane analogy and by relaxation methods. A circular section with a circular groove was investigated, the section being considered most suitable because an analytical solution is available and because it has a region of high stress comparable to the re-entrant corners of structural sections in which the authors are interested.

Both methods were found to be accurate to about 2 or 3 per cent. for this section, but each has a different field of use. For a complete investigation of the torsional properties, relaxation is considered the better method but the membrane analogy is equally effective when only particular constants are required.

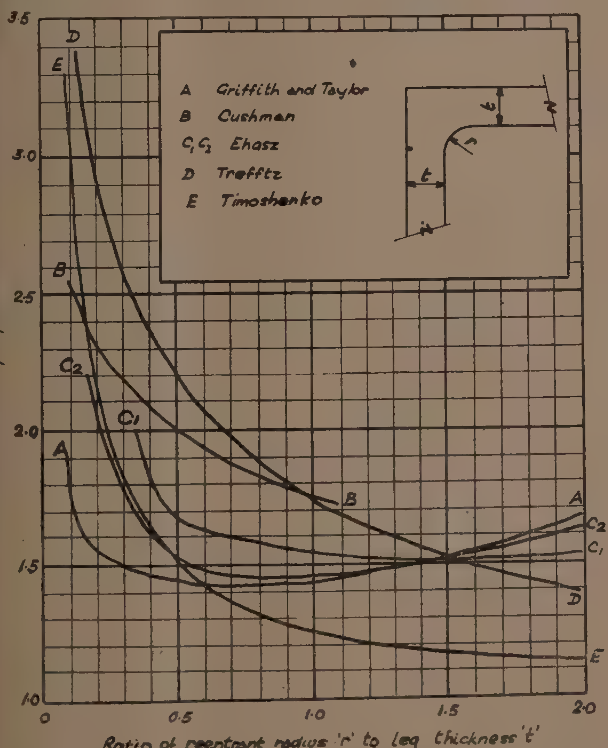


Fig. 1. Comparison of published results on stress concentration at the reentrant fillet of an equal-leg angle section

1. Introduction

The torsional properties of non-circular structural sections are generally determined, in design offices, by empirical formulæ based on the experimental results of a few investigators. It has, however, been shown in previous papers to the Institution[14, 15] that these formulæ for the torsion constant and the maximum elastic stress do not agree. Many of the variations can be attributed to the differences in the assumptions on which the empirical formulæ have been based, but it appeared that considerable error was also due to the basic experimental results.

This is demonstrated by the comparison of published results for the stress concentration at the re-entrant radius of an equal angle in Fig. 1, the ordinates of which represent the ratio of the maximum stress to that occurring in the leg of the angle at an infinite distance from the re-entrant corner. Curves D and E, due to Trefftz[16] and Timoshenko[2] respectively, represent formulæ derived by making general assumptions, but curves A, B and C represent the actual experimental results of Cushman[9], Ehasz[17], and Griffith and Taylor[8] respectively, obtained by the membrane analogy. Ehasz has given the results of tests on two similar angles, 4 in. $\times$ 3 in. $\times$ $\frac{1}{2}$ in. and 4 in. $\times$ 3 in. $\times$ $\frac{3}{8}$ in., which are given as curves C_1 and C_2 in Fig. 1.

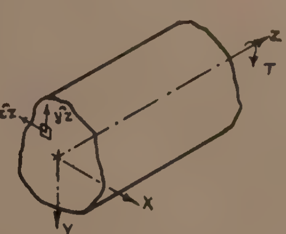


Fig. 2. Stresses due to pure torsion

Clearly, these results for the maximum stress are incompatible. This prompted an investigation into the accuracy of suitable methods for the determination of the torsional properties of structural shapes, in particular the torsion constant and maximum stress because of their practical interest.

2. Theory of Torsion

The equations governing the problem of pure, elastic torsion are based on the mathematical theory of elasticity and were first given by St. Venant[1]. Using the equations of equilibrium and the conditions of compatibility, the equation to be satisfied across the cross-section can be written[2], in terms of the stress function, ,

$$\nabla^2 \phi = -2 \quad \ldots \ldots \ldots \quad 1$$

where the operator is given by

$$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$$

The stresses $\widehat{xz}$ and $\widehat{yz}$, shown in Fig. 2, in terms of the stress function, are

$$\widehat{xz} = C\Theta \frac{\partial \phi}{\partial y}$$
$$\widehat{yz} = -C\Theta \frac{\partial \phi}{\partial x} \qquad \ldots\ldots\ 2$$

and the torque, T, is given by

$$T = 2\,C\Theta \iint \phi\ dx\,dy \qquad \ldots\ldots\ 3$$

where C is the modulus of rigidity and Θ is the twist per unit length.

At the boundary the additional condition

$$\frac{\partial \phi}{\partial s} = 0$$

must be satisfied, where s is the length along the boundary. This reduces to the condition

$$\phi = 0 \qquad \ldots\ldots\ 4$$

on the boundary, since the integration constant is arbitrary.

3. Methods of Solution

The possible methods of solving equations 1 and 4 can be divided into the following groups :

a. analytical methods,
b. methods of analogy, and
c. graphical methods and numerical computation.

Solutions obtained by direct mathematical analysis are confined to the simpler geometrical shapes, such as the circle and ellipse. By conformal transformation and the use of the complex variable more complicated shapes can be analysed, but these are limited to non-technical shapes.

The methods of analogy have been comprehensively surveyed by Higgins[3] ; they include :

a. Prandtl membrane analogy,
b. electrical analogies,
c. hydrodynamical analogies, and
d. photoelasticity.

In previous work, other investigators have favoured the membrane analogy using a soap film as a membrane, although Redshaw[4] and others have described and used electrical potential analysers for solving Poisson's equation. Analogies based on the hydrodynamical principle have never, to the writer's knowledge, been used for technical investigations. In an investigation into the stresses in keyways, Leven[5] has used photoelastic methods and concluded that the applicability of the scattering method to general cases of stress in regions of high stress concentration is yet to be demonstrated ; the surface method for obtaining surface stresses seemed to offer the best prospect. Photoelastic methods are still in the development stage and, although a solution of reasonable accuracy can be obtained, the cost of the models and equipment is rather high.

Many graphical methods of solution have been suggested, but in this group the most important way of obtaining a solution seems to be by numerical computation. There are two methods of computing a numerical solution, by

a. iteration, and
b. relaxation.

In iteration interest is centred on the value of the stress function and approach to the correct solution proceeds at a fixed rate, while in relaxation the rate of approach to the correct solution depends on the computer who is concerned mainly with the errors in the solution.

Of the many possible methods of effecting a solution, the writers favoured the membrane analogy and numerical computation by relaxation methods. They are both suitable for research work and yet require a minimum of intricate apparatus, and for this reason they could also be used in accurate design work. However, as neither of these methods gives a general solution their accuracy could be investigated only for a particular section.

4. Section Investigated

Having decided that the membrane analogy and numerical computation are favourable ways of obtaining solutions for complex sections it remained to find a section to the following specification :

a. an analytical solution must be available, and
b. the section must have at least one region of high stress such as occurs at a re-entrant corner.

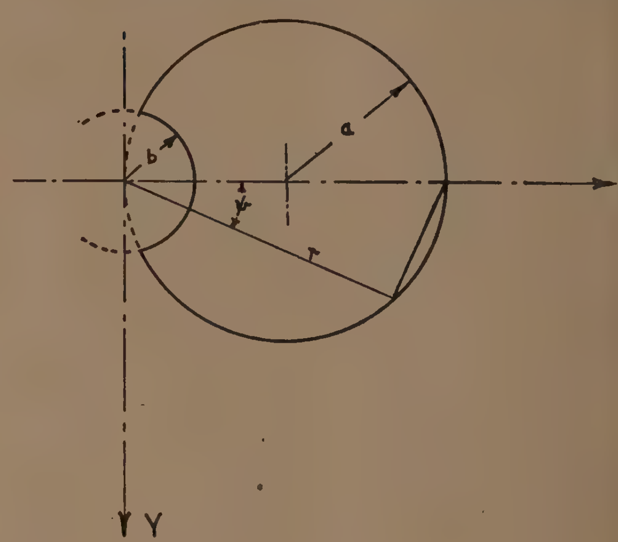


Fig. 3

The only section which seemed to fit this specification was the circular shaft with a circular groove, Fig. 3, the analytical solution being due to Weber[6] who gave

$$\phi = -C\Theta\left[\tfrac{1}{2}r^2 - ar\cos\psi + \frac{ab^2}{r}\cos\psi - \tfrac{1}{2}b^2\right] \qquad \ldots\ 5$$

From equation 5 the maximum stress, written as a stress factor, R, with the dimension of length, is

$$R = \frac{q_{max}}{C\Theta} = (2a - b) \qquad \ldots\ldots\ 6$$

and it occurs at the bottom of the groove. The torsion constant is

$$K = -2\left\{\left(\frac{a^4}{2} - a^2b^2 - \frac{b^4}{4}\right)\cos^{-1}\frac{b}{2a} + \tfrac{1}{2}\sqrt{4a^2 - b^2}\left(\frac{a^2b}{4} + \frac{7}{8}b^3\right)\right\} \qquad .\ 7$$

5. Membrane Analogy

For a membrane with a surface of double curvature it can be shown[7] that, approximately

$$\frac{1}{R_1} + \frac{1}{R_2} = -\frac{p}{S} \qquad \ldots\ldots\ 8$$

where R_1 and R_2 are the principal radii of curvature of the membrane, p is the applied pressure and S the tension per unit length of the membrane, which is assumed to be constant. Since the total curvature is invariant it can be written

$$\frac{\dfrac{d^2z}{dx^2}}{\left[1+\left(\dfrac{dz}{dx}\right)^2\right]^{3/2}} + \frac{\dfrac{d^2z}{dy^2}}{\left[1+\left(\dfrac{dz}{dy}\right)^2\right]^{3/2}} = -\frac{p}{S} \quad \text{. . . . . . 9a}$$

where z is the height of the membrane and x, y its plan co-ordinates. Higgins[3] gives the exact solution

$$\frac{\left[1+\left(\dfrac{dz}{dx}\right)^2\right]\dfrac{d^2z}{dx^2} - 2\dfrac{d\tau}{dx}\dfrac{d\tau}{dy}\dfrac{d^2\tau}{dxdy} + \left[1+\left(\dfrac{dz}{dy}\right)^2\right]\dfrac{d^2z}{dy^2}}{\left[1+\left(\dfrac{dz}{dx}\right)^2+\left(\dfrac{dz}{dy}\right)^2\right]^{3/2}} = \frac{p}{S} \quad \text{. 9b}$$

but, for the analogy, which requires that the inclination of the membrane be negligible compared with unity, these two equations reduce to

$$\frac{d^2z}{dx^2} + \frac{d^2z}{dy^2} = -\frac{p}{S} \quad \text{. . . . . . . 9c}$$

the analogy with the governing equation being self-evident. The boundary condition, equation 4, is satisfied by attaching the membrane to a hole in a thin plate, the shape of the hole being the same as the cross-section. Analogous phenomena are indicated in the following table :

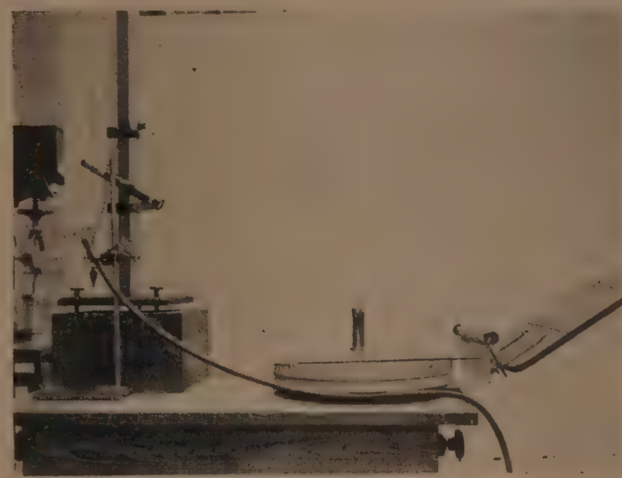
Membrane	Twisted Bar
Height, z	Stress function, $\varnothing$
Volume	Torque or Torsion Constant
Inclination	Stress

A soap film has, in the past, been used as a membrane and although attempts have been made to use other arrangements to avoid the difficulty of forming and maintaining a soap film, it remains the most efficient membrane. The relationship between the phenomena of the twisted bar and those of the membrane, termed the constant of the analogy, is a function of the applied pressure p, and the membrane tension, S. To avoid the direct determination of p and S, the membrane over the complex section is usually compared with one over a circle ; if the two membranes are made from the same soap solution and subjected to the same physical conditions the constant of the analogy is the same for both. In this way the torsional properties are determined, by proportion, from the known properties of the circular cross-section.

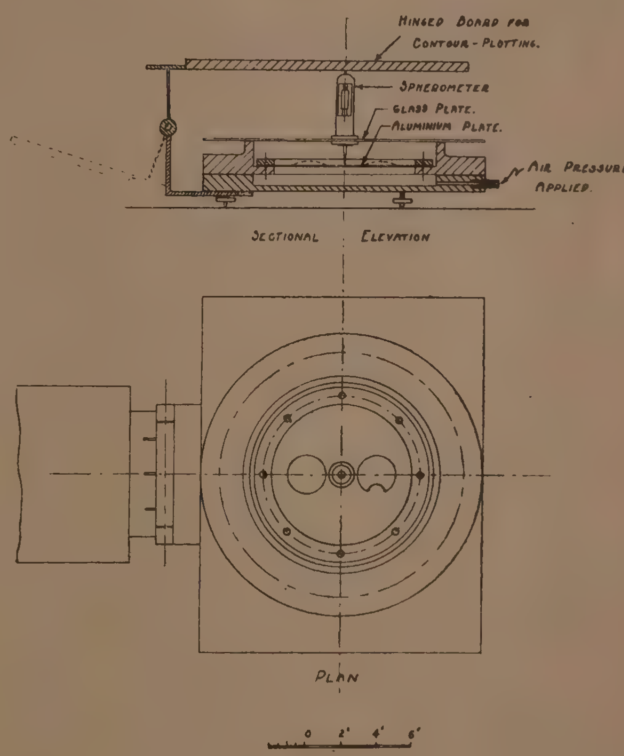
Fig. 4 shows the apparatus used which included a rigid airtight box to hold the plate on which the films were formed, a spherometer to determine the heights of the membranes and plot the contours, and an optical inclinometer to measure the inclination of the membranes. To measure small angles of inclination, the inclinometer was modified by fixing the light vertically above the point on the membrane for which the stress factor was required. The volume enclosed by the membrane can be determined from a contour diagram but it was found more efficient to determine the volume directly by inflating the membranes with a U-tube and burette.

5.1 Torsion Constant

The torsion constant, K, is proportional to the volume under the membrane. Assuming a spherical film over the circle, the volume, V_c, can be computed from its height, h, and the corresponding volume, V_t, for the non-circular section determined by measuring the sum $(V_t + V_c)$. The films were inflated by the U-tube, one leg of which was connected to the apparatus and into the other, water was added from a burette. Neglecting the slight pressure losses due to the compression of the air in the apparatus, the increases in the sum of the volumes were predetermined. The height, h, was measured for a given increase in $(V_t + V_c)$.



(a)



(b)

Fig. 4. Soap film apparatus

In terms of the height, h, the volume in a spherical film is

$$V_c = \tfrac{1}{8}\,\pi\,h\,D^2 \left[1 + \frac{4}{3}\,\frac{h^2}{D^2}\right] = \tfrac{1}{8}\,\pi\,h^1 D^2 \quad \text{. . . 10a}$$

where $h^1 = h\left[\dfrac{4}{3}\,\dfrac{h^2}{D^2}\right]$

and D is the diameter of the circular hole. Fig. 5 shows the graph of Δh^1 vs. $2\,\Delta\,(V_t + V_c)$; the torsion constant

calculated by the method of least squares from these results was 1.740 in⁴ which is in error by —2.69 per cent.

For small h/D, equation 10a approximates to the expression for the volume, V_{cP}, enclosed by the Prandtl membrane over a circular hole of the same diameter D, viz.,

$$V_{cP} = \tfrac{1}{8}\,\pi h D^2 \qquad \text{10b}$$

The graph of Δh vs. $2\,\Delta(V_c + V_t)$ can therefore be used to compute the torsion constant : the points are shown in Fig. 5. Using the method of least squares and neglecting the effect of $(h/D)^2$, the torsion constant was found to be 1.825 in^4, which is in error by +2.06 per cent.

Griffith and Taylor[8] in their experiments used an inclinometer in which the incident and reflected rays of light coincided. The films were inflated and the slopes determined by adjusting the inclinometer. The writers considered that results with comparable accuracy could be obtained quicker by the method outlined above.

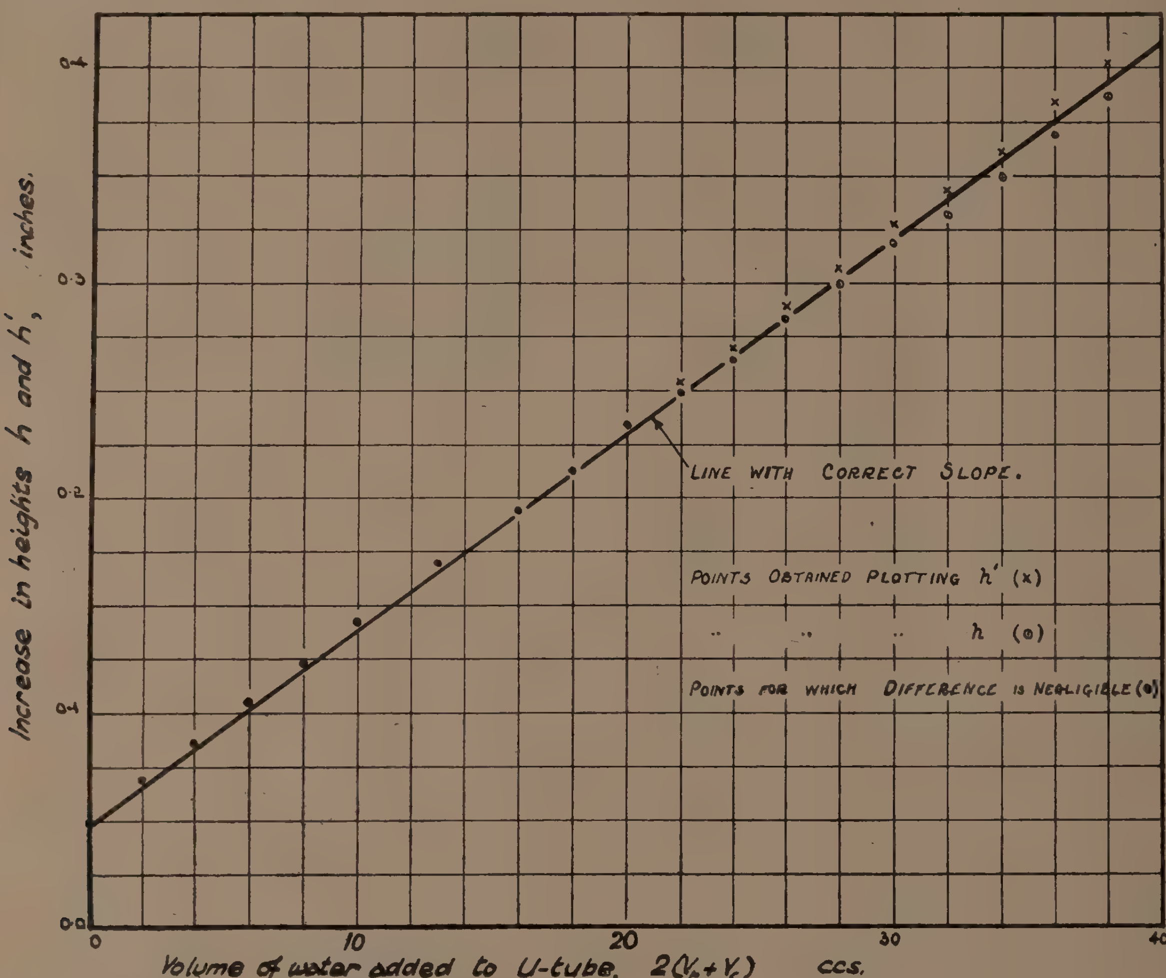


Fig. 5. Correlation of volume with height of membrane

5.2 Maximum Stress

The determination of the stress factor, which is proportional to the slope of the Prandtl membrane, requires the measurement of the inclinations at the boundary of the circle and at the required point of the complex section. The inclination, i_t, of the complex section was measured by the optical inclinometer which records by reflection ; for the circle it was assumed that the film was spherical and the inclination, i_c, was computed from its height, h, measured by a spherometer.

The telescope of the inclinometer was first adjusted so that its line of sight passed through the point of maximum stress for the complex section. The light was then fixed in a specified position and the films inflated until the reflection of the light source appeared on the membrane. The membranes were then slowly deflated until the reflection of the light source sank below the centre of the cross-hairs of the telescope. The maximum height of the film over the circle was then recorded.

5.21 Edge Effect

When using the optical inclinometer an edge effect on the soap film was observed which affected the accuracy of individual measurements. It appeared as a dark rim at the edge of the plate, when viewed through the telescope, as shown in Fig. 6. The reflection of the light source was not visible below point B in Fig. 6 and at B there was a pin-point reflection of the light source which did not move appreciably as the films were

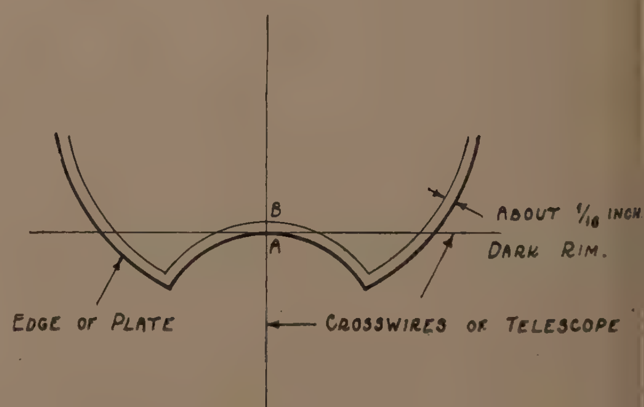


Fig. 6

lated and which remained after the normal reflection 1 disappeared. The apparent height of the dark rim, ?, varied but had a maximum of about 0.0625 in.

Cushman[9] suggested that the edge effect was due to film leaving the edge of the plate, as in Fig. 7a, the face of which was always wet due to the method of ming the films. It was observed however, that when film was inflated high enough to cause a visible vement over the plate surface, the apparent edge the plate was distorted relative to the cross-hairs due refraction as in Fig. 7a. As no optical distortion was

Fig. 7. Edge of soap film

served in normal tests the edge effect cannot primarily caused by the film adhering to the plate surface stead of the edge. Probably the edge effect was due liquid which had drained down the film and collected und the edge of the plate as in Fig. 7b. This would count for the observed increase of edge effect with a icker film, with the surface tension of the solution, hich would increase the amount of liquid that could held in the rim, and with time. The sudden dis- pearance of the reflection at the top of the edge effect, in Fig. 6, can be interpreted as the point at which the rvature changes from convex to concave. There pears to be no simple explanation of the point reflec- on at B but it is possibly due to complicated internal flections within the soap film.

When measuring the slope of the membrane Cushman[9] ed the spherometer in an attempt to overcome the fficulty caused by the edge effect. He measured the ight of the film at intervals from the edge of the plate d obtained the slope by extrapolation assuming the edge effect was local. The method was considered, by the writers, to be less accurate than the optical inclinometer because any slight errors in the height measurements would be proportionally large when determining slopes.

5.22 Analysis of Results

As the stress is theoretically proportional to the tangent of inclination of the Prandtl membrane, the results were plotted as a graph of tan i_t against tan i_c. The graph was not a straight line but a smooth curve approaching the theoretical line at the origin, as indicated for a particular soap solution in Fig. 8. The deviation from the straight line is attributed to the inaccuracy of the analogy which only holds for small angles of inclination.

Griffith and Taylor[8] plotted *sin* i_t *vs. sin* i_c and assumed that the graph was a straight line giving the ratio between the stress factors for the complex section and the circle. When, however, our results, obtained with many different soap solutions, were plotted in this way a graph with an error of about 10 per cent. was obtained, Fig. 9.

As greater accuracy was required a correction formula was devised for the graph of *tan* i_t *vs. tan* l_c. The formula was deduced by considering the film over the circle and it was extended to other shapes. The Prandtl membrane is parabolic and analogous to the solution of the general torsion equation, viz.,

$$\varnothing = \tfrac{1}{2}(R^2 - r^2) \quad \ldots \ldots \quad 11$$

The soap film is known to be spherical and therefore for any film of height h the corresponding Prandtl membrane can be visualised as in Fig. 10. A relation clearly exists between the measured slope, *tan* i_c, and the slope *tan* $i_c{}^1$ of the Prandtl membrane. From geometry this relationship is given by the quadratic equation

$$\tan^2 i_c{}^1 - \frac{4}{\tan i_c} \tan i_c{}^1 - 4 = 0 \quad \ldots \ldots \quad 12$$

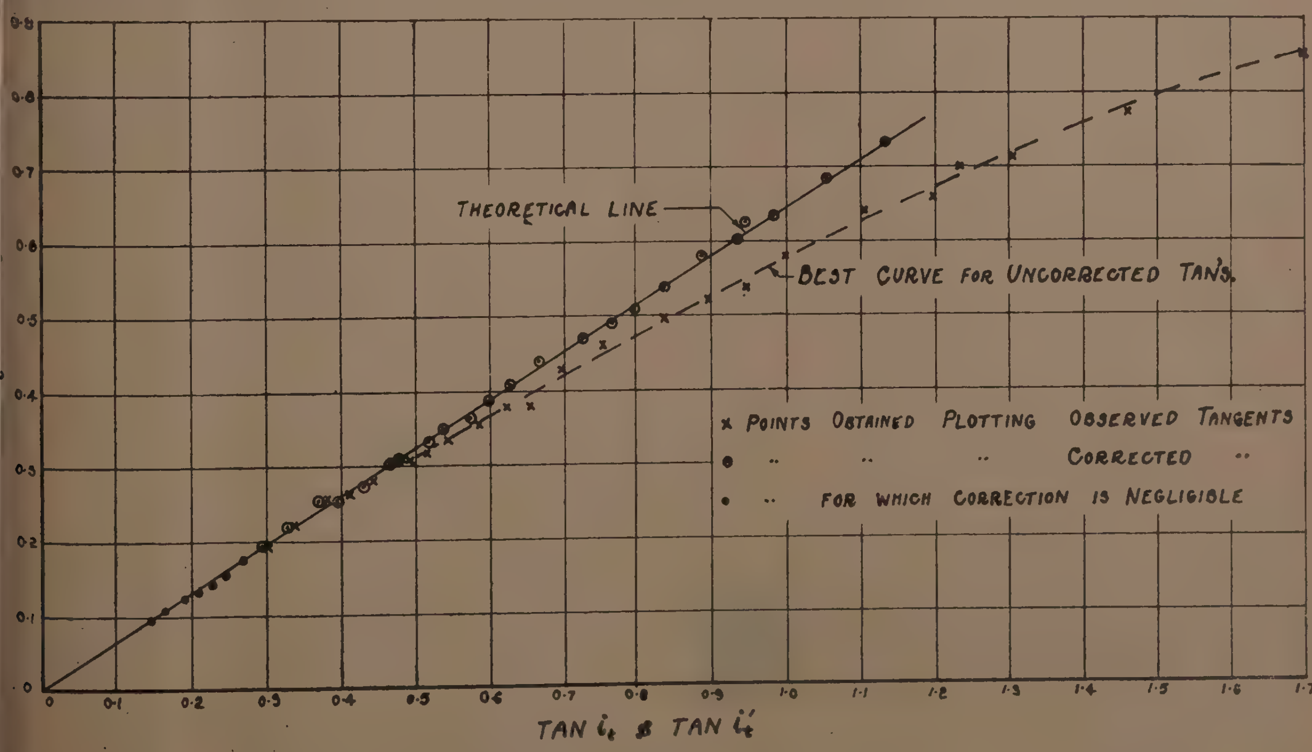


Fig. 8. Comparison of observed and corrected tangents

A graph of this equation, Fig. 11, was used to adjust the observed values of *tan* i_e to the corresponding *tan* i_e^1. The expression was also applied to non-circular sections, since at any point their soap films can be considered spherical. Correcting the graph, Fig. 8, a good straight line was obtained with an error in the stress factor of only —1.71 per cent.

Thick viscous solutions generally gave durable film but they were difficult to form over large holes and we accompanied by considerable edge effects.

5.32 Size of Circle

Griffith and Taylor[8] showed that, for the mean si of the inclination of the test film at the boundary to b

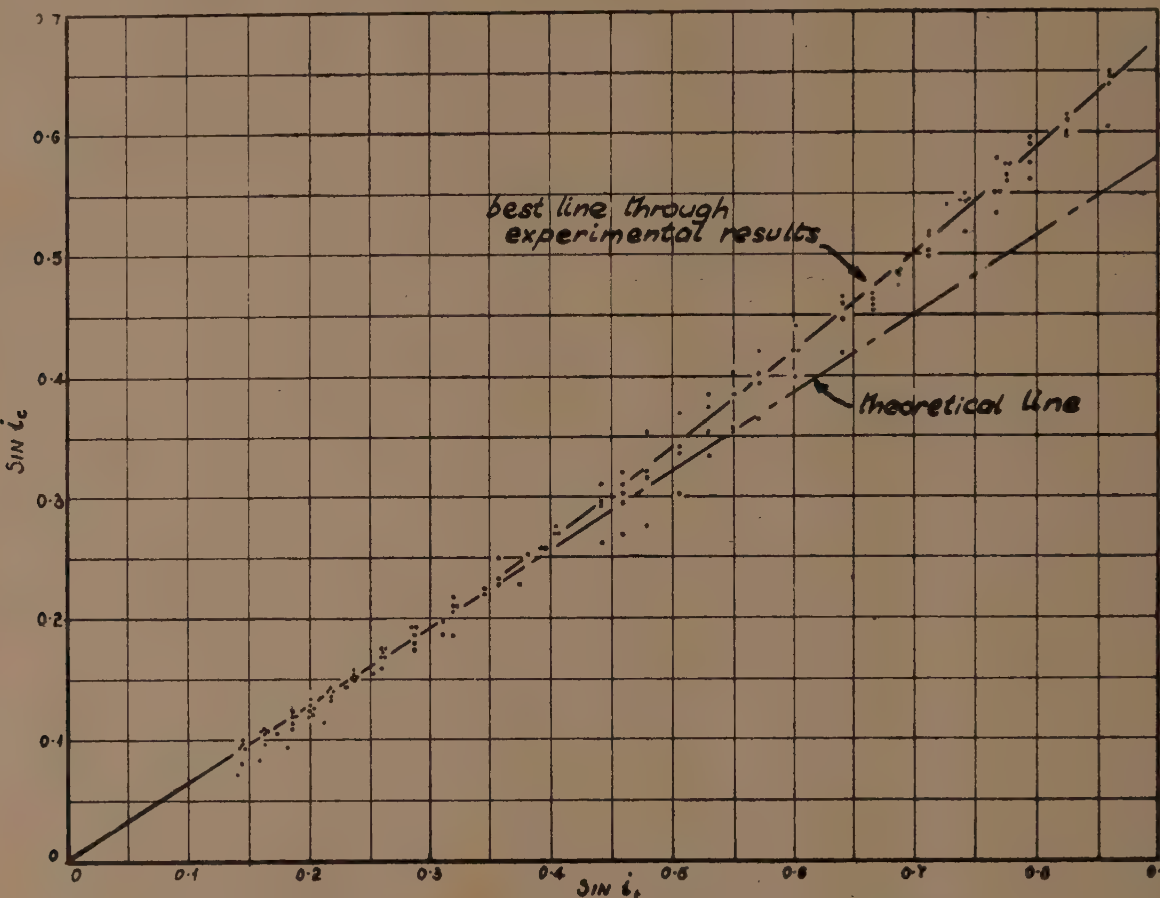


Fig. 9. Graph of sines of inclination showing deviation from straight line

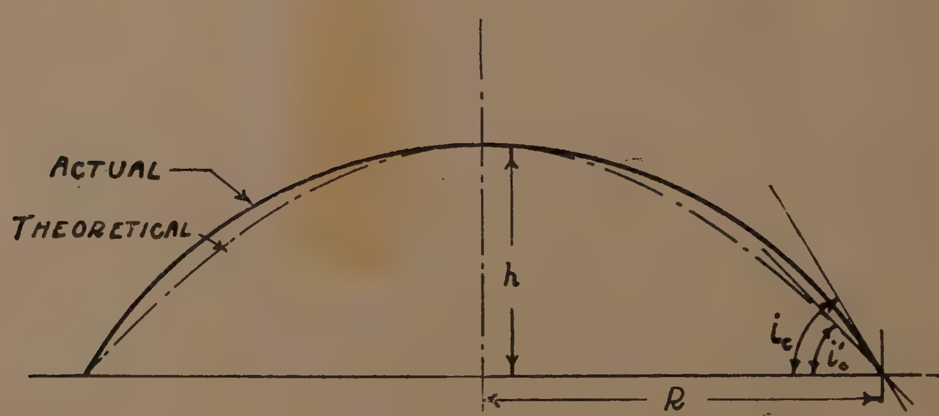


Fig. 10

5.3 Conclusions on Membrane Analogy

5.31 Soap Solution

For stress determinations a soap solution(a) is required which gives the minimum edge effect, durability being a secondary consideration. A soap solution(b) producing a good durable film is, however, required when estimating the torsion constant because of the procedure suggested in which all the measurements are made with one film.

equal to the sine of inclination of the film at the bounda of the circle, the circle should be of radius $2A/P$ where

a. 15.2 cc. of Oleic acid was mixed with 50 cc. of distil water and 7.3 cc. of 10 per cent. aqueous solution of triethano mine added. This was then made up to 200 cc. with distil water and 16.4 cc. of pure glycerine added. The solution w allowed to stand in a separating funnel and the clear liquid dra off from below.

b. 10 grammes of sodium oleate and 50 cc. of pure glyceri made up to 500 cc. with distilled water.

s the area and P, the perimeter of the test section. circle of this radius was recommended and it is sonable for torsion constant measurements since it ggests that the degree of approximation in the analogy, e to the neglect of $\frac{\partial z}{\partial y}$ and $\frac{\partial z}{\partial x}$ in equation 9 is the ne for both films. For stress measurements, however, condition is not as reasonable ; a better condition to ow for the error in the analogy is to adjust the radius the circle so that the inclination of the film at the undary is approximately equal to the inclination of e test film at the point where the stress measurements being made.

Further, the size $2A/P$ is sometimes inconveniently all and the errors made in determining the film ight then outweigh the advantage gained in making

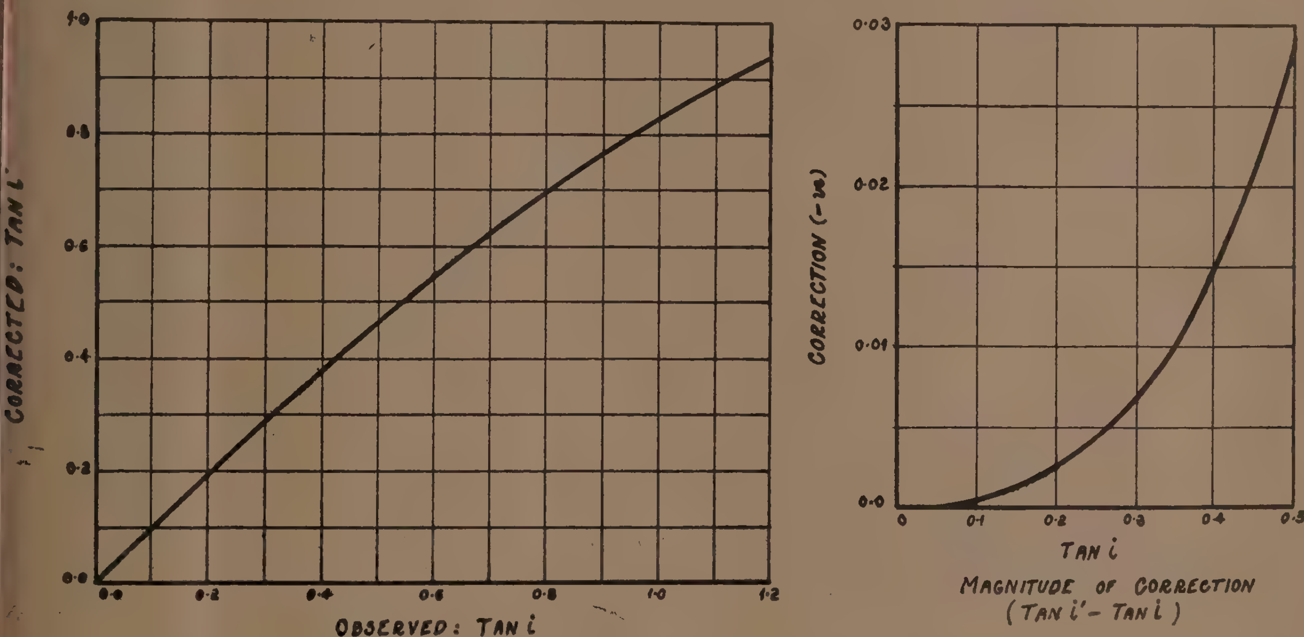


Fig. 11. Graphs for determining corrected from observed tangents using correction formula $\tan^2 i' + 4 \tan i'/\tan i - 4 = 0$

qual the analogy approximation for both films. The rrors arise because the accuracy of measuring the film eight is proportionally large for small heights. The ircle should not, therefore, be less than about 1.5 in., ut care must be taken to ensure that the ratio of the olumes enclosed in the films, V_c/V_t, is not excessive, therwise one encounters difficulty in determining the mall volume V_t by difference.

·33 Stress Determinations

It is recommended that the ratio between the stresses the test and circular sections be determined by lotting the tangents of the inclinations of their films, fter correction by Fig. 11. This correction makes an ttempt to allow for the inaccuracies in the analogy nd is based on the known relationship between the soap lm over a circular hole and the stress function for a ircular section ; there is little danger of erroneously pplying the correction formula as the result would be nother curve instead of a straight line. If the diameter f the circle used for comparison had been adjusted so hat the values of i_t and i_c were equal, the correction for both would have been equal and there would have been no need to correct the observed results. Fig. 10 demonstrates the inadvisability of using a sine plot, as recommended by Griffith and Taylor[8], to determine the stress ratio.

5·34 Measuring the Inclination of the Films

Results with comparable accuracy can be obtained quicker by slowly deflating the films to a particular inclination than by measuring the inclination by adjusting the optical inclinometer. Further, the method allows a personal estimation of the edge effect.

Cushman's method of estimating the inclinations using the spherometer is not recommended, principally because the method is essentially a finite difference one and depends on the intervals between the very small height measurements which are themselves of doubtful accuracy.

5·35 Accuracy of the Torsion Constant and Maximum Stress Determined by the Membrane Analogy

The following table gives the results obtained by the soap film apparatus for a section in which

a = 1.117 in. nominally 1.125 in.
b = 0.482 in. nominally 0.500 in.

	Membrane Analogy	Theoretical	Error
q/CO max	1.722	1.752	−1.71%
K	1.825	1.788	+2.06%

6. Numerical Methods

Analytical methods are not applicable to problems involving highly irregular regions and in some cases the equipment is not available to carry out analogous

experiments. This necessitates the use of graphical or numerical methods for complex shapes.

The wider use of numerical methods in all branches of Applied Physics has made graphical methods appear outdated. There are two types of numerical methods :

a. the method of iteration, originally associated with the name of Liebmann[10] and used by Orr and Thom[11] for the torsion of circular shafts of varying radius and later by Orr[12] for an investigation into the torsion of structural shapes, and

b. relaxation methods due to Southwell[13] which is a form of end-figure tabulation.

In both cases, the differential equations of the torsion problem must be written in finite-difference form, which reduces the problem to one with a finite number of degrees of freedom. The function $\Delta^2\phi$ has a particular value for every point in the domain in which we are

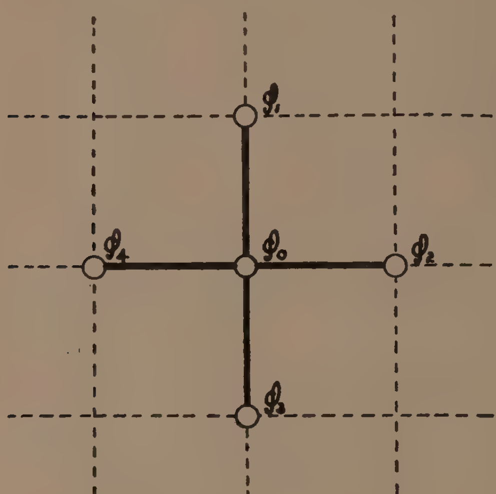


Fig. 12

interested but, by interpolation formulæ, the value of this function could be estimated from certain known values at specific points. Such specific points are usually arranged in an array, the continuum of the original body being considered as a network of these points.

Using Liebmann's approximation for the Laplacian operator, the finite difference form of the governing equation for the square net, Fig. 12, is

$$\frac{1}{a^2}\left(\sum_{n=1}^{4}\phi_n - 4\phi_0\right) + 2 = 0 \quad . \; . \; . \; . \; . \quad 13$$

A new improved value of ϕ_0 at any specific point can be computed from equation 13 using assumed values of the stress function at the specific points or nodes of the network. This constitutes the basis of the iterative method. Values of ϕ are assumed to begin with and using the equation

$$\phi_0 = \tfrac{1}{4}\left(\sum_{n=1}^{4}\phi_n + 2a^2\right) \quad . \; . \; . \; . \; . \; . \; . \; . \quad 14$$

each one is corrected in turn. Convergence to the correct solution proceeds at a fixed rate beyond the control of the operator and for this reason the writers preferred the relaxation method.

In the relaxation method, values of the stress function are assumed as before and corrected by a modified form of equation 13. The error in the assumed values is measured by the right-hand side of equation 13 whic now becomes the residual, F_0.

$$\sum_{n=1}^{4}\phi_n - 4\phi_0 + 2a^2 = F_0 \quad . \; . \; . \; . \; .$$

It is obvious that the following changes in the stre function cause corresponding changes in the residua

$$\Delta\phi_n = +1 \qquad \Delta F_0 = +1$$
$$\Delta\phi_0 = +1 \qquad \Delta F_0 = -4$$

By suitably adjusting the values of the stress functio the computer can reduce the residuals to values which l considers negligible. Experience of the relaxatic procedure, adequately described in the publications Sir Richard Southwell and his co-workers, enables tl computer to increase the speed of convergence to tl desired solution. Practice of the relaxation methc on a simple section, such as a square, will demonstra this fact.

The finite difference form of the governing equatio for the irregular parts of the network at the bounda is given by Southwell[13] ; procedures for advancing to finer net are also given.

Fig. 13 shows the finished network for the circul section with a circular groove. Initial assumed valu were obtained by the membrane analogy but the could equally well have been assumed from the solutic for a circle by sketching the contours. The origin side of the network was 0.5 in. and the residuals we reduced to about 2 per cent. of the stress function value The network was then reduced, by stages, to a fin size of 0.0625 in. over most of the area.

Stress factors for the re-entrant surface, which a proportional to the slopes $\frac{\partial\phi}{\partial n}$, were computed fro linearly interpolated values of the stress function. Tl slope $\frac{\partial\phi}{\partial n}$ was determined by assuming that a secor degree polynomial described the dependence of ϕ c the normal distance, n. Values of the stress factor, computed, are compared with the theoretical curve Fig. 14 ; for nodes very close to the boundary, erroneo results were obtained and it is recommended that tl complete distribution curve should be drawn whe evaluating the stress at any point on a re-entra surface.

An isolated computation of the stress factor for th network of 0.0625 in. side gave a value of 1.696 whi is in error by 3.08 per cent. By reducing the netwo to 0.03125 in. over a small area adjacent to the poi of maximum stress, Fig. 13a, the error was reduced 2.17 per cent.

The torsion constant, K, which is given by

$$K = 2\iint \phi \, dx \, dy$$

can be determined in two ways :

a. by drawing lines of constant stress function, determining the area, A_ϕ, enclosed by them and the estimating the value of $\int A_\phi \, d\phi$ by Simpson's rule,

b. by summing the values of ϕa^2 for each node.

For the size of network illustrated, both methods are equal accuracy but the latter is quicker. The torsic constant in both cases was found to be 1.812 in^4 whi is in error by 1.79 per cent. A network of 0.0125 i

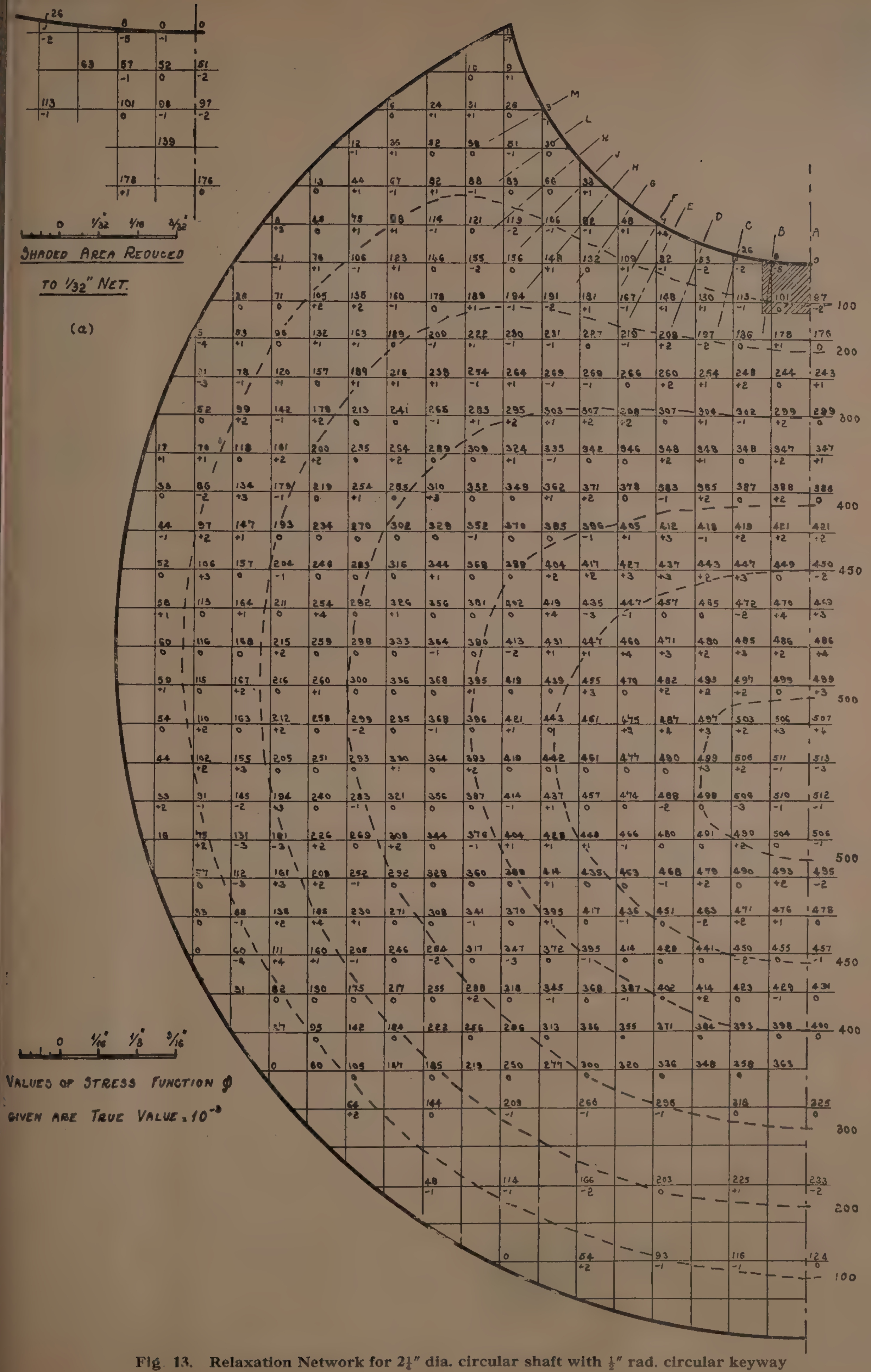


Fig. 13. Relaxation Network for $2\frac{1}{4}''$ dia. circular shaft with $\frac{1}{2}''$ rad. circular keyway

side would no doubt give a torsion constant of sufficient accuracy.

7. Conclusions

Summarising the results of the investigation,

	Membrane Analogy a 1.117″; b 0.482″			Relaxation Method a 1.125″; b 0.500″		
	theory	experiment	error	theory	experiment	error
$q/C\theta$max	1.752″	1.722″	1.71%	1.750″	1.712″	2.17%
K	1.788in^4	1.825in^4	2.06%	1.845in^4	1.812in^4	1.79%

Acknowledgement

The work was carried out in the Department of Civil Engineering, King's College, Newcastle-upon-Tyne, and formed part of a programme of research on Torsion in Structures directed by Professor W. Fisher Cassie.

References

[1]B. de St. Venant, "Memoire sur la torsion des Prismes," Mem. des Savants Etrangers, 14 (1855) 233.

[2]S. Timoshenko, "Theory of Elasticity," McGraw Hill, 1st edition (1934) chapter 9.

[3]T. J. Higgins, "Analogic experimental methods in stress analysis as exemplified by St. Venant's torsion problem," Proc. of Society for Experimental Stress Analysis, 2.2 (1944) 17.

[4]S. Redshaw, "An electrical potential analyser," Proc. I. Mech. E., 159 (1948) 55, War Emergency Issue No. 38.

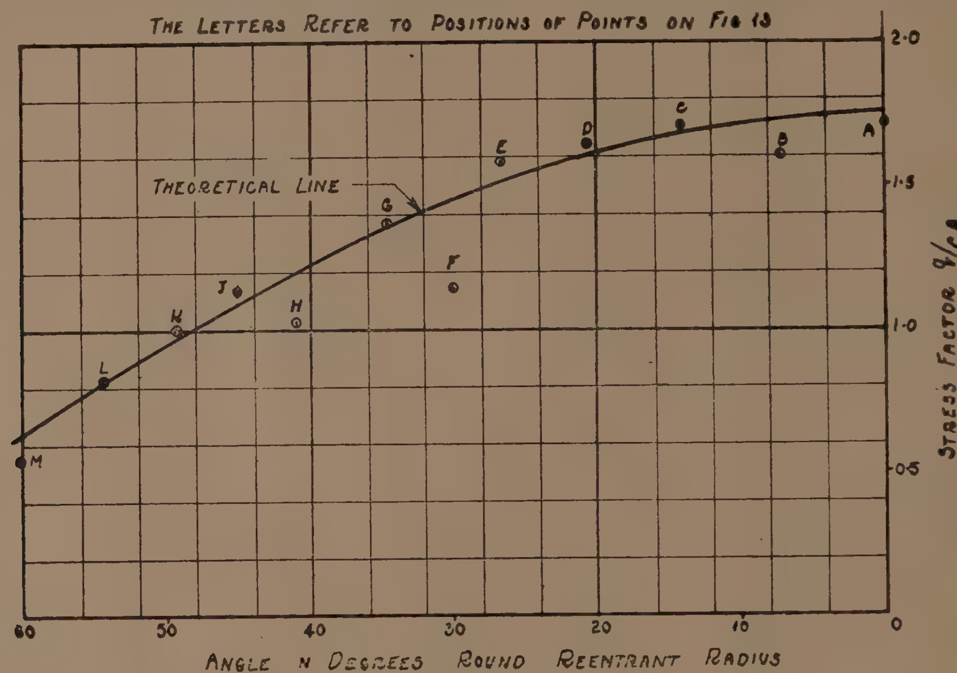


Fig. 14. Stress Distribution round reentrant radius

From the table one sees that the error for both methods is of the order of 2 per cent. Certain precautions are however necessary if this accuracy is to be obtained, as follows.

Membrane Analogy

a. A suitable soap solution must be used which gives minimum edge effect and is sufficiently durable, and

b. the circle used for comparison should not be less than 1.5 in. diameter.

Relaxation Methods

a. Residuals should not be greater than 2 per cent. of the corresponding stress function value, and

b. the size of the network in the region of a re-entrant corner should be about one-eighth of the re-entrant radius.

Instead of plotting sines of the membrane inclinations to get the stress factors as suggested by Griffith and Taylor,[8] it is recommended that the measured tangents be adjusted by the correction formula given in art. 5.22 and illustrated in Fig. 11.

For re-entrant radii less than about 3/16 in. relaxation methods are superior to the membrane analogy, but both methods are generally suitable for determining the properties of structural sections from which empirical equations can be derived. Comprehensive tests[14] on structural sections, using the two methods discussed, showed that similar accuracy could be obtained for other sections.

[5]M. M. Leven, "Stresses in keyways by photoelastic method and comparison with numerical solution," Proc. of Society fo Experimental Stress Analysis, 7.2 (1950) 141.

[6]C. Weber, "Die Lehre der Drehungsfestigkeit," Forschung d. Gebiete d. Ing., 249 (1921) 1.

[7]E. Edser, "General Physics for Students," Macmillan, 1s edition (1922) 322 sq.

[8]A. A. Griffith and G. I. Taylor, "Use of soap films in solvin torsion problems," Proc. I. Mech. E. (1917), 755.

[9]P. A. Cushman, "Shearing stresses in torsion and bending b the membrane analogy," unpublished paper No. 38 of A.S.M.E (1932).

[10]H. Liebmann, "Die angenäherte Ermittlung harmonische Funktionen und Konformen Abbildungen," Sitz. d. Math. un Phys., 47 (1918) 385.

[11]J. Orr and A. Thom, "Torsional problem of shafts of varyin radii," Proc. Roy. Soc., Series A, 131 (1931) 30.

[12]J. Orr, "Torsional properties of structural and other sec tions," Proc. I.C.E., Selected Engineering Paper No. 128, (1932

[13]R. V. Southwell, "Relaxation Methods in Theoretic Physics," Oxford U.P., 1st edition, (1946) ; see also "Relaxatio methods as applied to structures," STRUCTURAL ENGINEER, 26. (1948) 463, and "Relaxation methods : an engineering approac to computation," Proc. I.C.E., 30.8 (1948) 351. First publishe for the torsion problem by D. G. Christopherson and R. V Southwell, "Relaxation methods applied to engineering prol lems, III ; problems involving two independent variables, Proc. Roy. Soc., Series A, 168 (1938) 317.

[14]W. B. Dobie, "The torsional strength of structural members," THE STRUCTURAL ENGINEER, 30.2 (1952) 34.

[15]W. F. Cassie and W. B. Dobie, "The torsional stiffness of structural sections." THE STRUCTURAL ENGINEER 26.3 (194 154.

[16]E. Trefftz, "Uber die Wirkung einer Abrundung auf d Torsionspannungen in der inneren Ecke." Z.a. M.M. 2 (1922) 26

[17]F. L. Ehasz, Discussion of paper "Structural Beams Torsion," Proc. A.S.C.E. 61 (1935) 1222.

The Fire Endurance of Timber Beams and Floors*

Discussion on the Paper by D. I. Lawson, M.Sc., M.I.E.E., C. T. Webster, F.R.I.C. and L. A. Ashton

The CHAIRMAN proposed a vote of thanks to the authors and declared the meeting open for discussion.

Mr. A. S. PRATTEN, London Salvage Corps, asked whether results might vary with the nature and age of the wood. He had particularly in mind the difference between soft and hard wood.

Mr. Ashton replied that the question was very wide in scope and they had not done enough experimental work to answer it fully. But the small amount of work they had done on charring timber indicated that there was not a significant difference in the rate of charring between soft and hard wood under the conditions of their tests, and they therefore would not expect much difference in the fire endurance of beams made from hard or soft wood.

With regard to the age of the wood, they would not like to make any prediction because they had not been able to explore that factor. There was one factor which was important and which had not been mentioned, the moisture content. Their timber had been conditioned to a moisture content of about 12 per cent., which was common in buildings.

Mr. S. J. DOCKING said he was indebted to the authors, who had been war-time colleagues, for exploring a problem which was not just an occasional one, but concerned every-day building.

In our modern research for substitute materials for construction, the question of fire resistance was often overlooked in the effort to achieve maximum economy in the single factor of load bearing. It was obvious that a square beam, which was generally an impracticable shape, was the best for fire resistance. What he had found interesting in the paper was the evidence given in Fig. 6 that shapes which were structurally economical were not entirely ruled out of court from the fire-resistance point of view. He was pleased that the necessary degree of fire-resistance could be met by providing a good ceiling to give protection to the members rather than by spoiling the structural shape of the joists themselves.

Going from that to a point beyond the paper, he had been much struck in his own studies on fire resistance by the relative weakness of steel members as compared with timber members. It was strange that the combustible material would often stand up to a fire and carry its load better than an incombustible material doing the same job. For that reason he hoped the authors would go forward to study the comparative fire susceptibility of such things as welded steel joists and those other light forms of beam which we had been driven to adopt because of the shortage of timber. He thought it would be found in some cases that the fire susceptibility was so high that especially good ceilings ought to be used in certain cases ; for example, under bedrooms over living-rooms, where there was a "sleeping" risk.

He thanked the authors for lifting the question of fire susceptibility out of the slow trial and error approach which was sufficient for the leisurely days of the past into the scientific approach which we needed now to cope with rapid developments of modern building technique.

Dr. S. B. HAMILTON (Vice-President) called attention to the assumption tentatively made by the authors in the second paragraph on p. 27, "that the timber retains its original strength until charring takes place." Since hot, damp wood is known to be much weaker than cold, dry wood, this assumption could not have been expected to be even roughly true and was indeed by implication abandoned later by the authors. They did not, however, as they could have done from their experimental results, check the degree to which the assumption was erroneous. When this assumption is abandoned, k in equation (1) is not a constant, and the two expressions stated in equation (2) to have the same value r are not equal.

The r which has the value expressed in the second line of equation (2) and in equations (3) and (4) is actually the ratio between the modulus Z_t of the section of the beam EFGH in Fig. A to the modulus Z_0 of the original

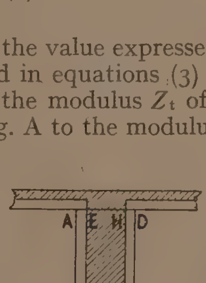


Fig. A

section ABCD when the unhatched area between the boundaries represents the material made useless by charring. The r which is the ratio between W_t and W_0 is found by curve-fitting to have the value shown in equation (7). It is this value of r which is used by the authors in all subsequent references in tables and graphs. It would have been clearer had different symbols been used for the two r's defined thus

$$r_1 = \frac{Z_t}{Z_0} \text{ and } r_2 = \frac{W_t}{W_0} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

Since W_t is applied to the beam at the beginning of the test, it remains unaltered until failure and so does the moment M_t, under which failure takes place after t minutes endurance. If the modulus of rupture of the

*Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 14th, 1952, at 6 p.m. Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair.

beam at failure under fire test is f_t, then $M_t = f_t.Z_t$; and if the modulus of rupture of a similar beam tested to failure, cold, is f_u, taken by the authors as 11,000 lb./sq. in., and the moment at failure for the cold beam is M_0, then

$$r_2 = \frac{W_t}{W_0} = \frac{M_t}{M_0} = \frac{f_t.Z_t}{f_u.Z_0} = \frac{f_t.r_1}{f_u} \quad . \; . \; . \; . \; (9)$$

By referring to "moduli of rupture," and not to extreme fibre stresses at failure loads, any assumption as to the distribution of stress over the section at failure is avoided.

The relationship between r_2 and r_1 in equation (9) may now be compared with that implicit in the authors' equation (7).

$$\left.\begin{aligned} r_2^{\frac{1}{2}} &= \tfrac{1}{2}.r_1, \\ \text{or } r_2 &= \frac{r_1^2}{4} \end{aligned}\right\} \quad . \; . \; . \; . \; . \; . \; . \; . \; (7)$$

$$\text{thus : } \frac{r_1^2}{4} = \frac{f_t}{f_u}.r_1$$

$$\text{From which } f_t = \frac{f_u}{4}.r_1 = 2750.r_1 \quad . \; . \; . \; . \; . \; (10)$$

Taking as an example the test results given in Table 1 for beams 9 in. by 2 in., r_1 varies from 0.57 when $t =$ 15 min. to $r_1 = 0.19$ when $t = 29$ min. The corresponding variation of f_t is from 1570 to 520 lb./sq. in.

Equation (7) is reasonably true for the range of the experiments, that is for times between 10 and 30 minutes and for shape factors ($S = \frac{d}{b}$) between, say, 2 and 6 ; but to extrapolate beyond these limits is to invite serious error. For instance, for low values of t the weakening of the bulk of the timber by heating may be slight. For $t = 0$, $r_1 = 1$ and by equation (10) f_t would be 2750 ; but actually when $t = 0$, $f_t = f_u = 11{,}000$. The family of curves plotted in Fig. 6 goes far beyond the limits of experimental evidence in respect to S ; only about the middle third of the range plotted is actually so backed. The authors' conclusion therefore (on p. 33) that "the maximum fire-endurance in relation to the cross-sectional area will be achieved for beams of approximately square cross-section when loaded to a given fraction of its breaking load," since it is based on extrapolated curves, cannot be regarded as proved. Further experiment might show it to be incorrect. That this conclusion, if accepted tentatively, may be misleading appears from the following comparison.

The two beams, of the sections shown in Fig. B both have the same cross-sectional area, $a = 16$ sq. in., but for the deep beam $S = 4$ and for the square beam $S = 1$. Let each be loaded to one-tenth of its breaking load, that is, let each sustain when cold an extreme fibre stress of 1,100 lb./sq. in. for which $r_2 = 0.10$. Then $r_1 = 2\sqrt{r_2} = 0.633$ and $f_t = 2750r_1 = 1740$; both beams will fail at the same modulus of rupture, but after different times of endurance. The values of t which will satisfy equation (3) are—for the deep beam 12.5 minutes and for the square beam 16.8 minutes.

In Table Z, columns (2) and (3) it will be noticed that though the deep beam fails a few minutes earlier than the square beam when equally stressed, it carries at the stresses assumed twice the moment, or load, on the square beam. To carry the same load at the same

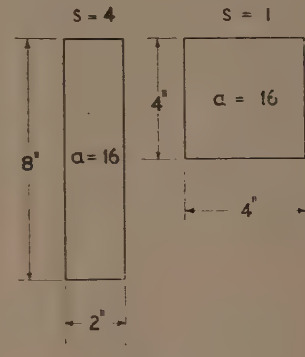


Fig. B

stresses, two square beams would be required embodying twice as much timber as one deep beam. If a deep beam and a square beam were used to carry the same load with the square beam stressed as in column (3), the deep beam as shown in column (4) would sustain when cold an extreme fibre stress of only 550 lb./sq. in., or one twentieth of its breaking load. The endurance of the deep beam would then be 19.6 minutes, which gives the deep beam an advantage of nearly three minutes in fire endurance over the square beam. It thus appears that the Conclusion drawn in Section VIII of the paper would have been better omitted. So far from adding to the really valuable results of the authors' work embodied in Table V and in Figs. 7 and 8, it merely introduces confusion.

TABLE Z

A comparison between the properties and fire-endurance of two beams of equal cross-section, a, and unequal ratios of depth to breadth, S.

	Equal Stresses		Equal Loads
(1)	(2)	(3)	(4)
Section (a = 16 sq. in.)	8 in. × 2in.	4in. × 4in.	8 in. × 2in.
S	4	1	4
Z_0	21.33	10.67	21.33
Working stress, f_w ...	1,100	1,100	550
Applied Moment $M_t = f_w.Z_0$	23,500	11,700	11,700
r_2	0.10	0.10	0.05
$r_1 = 2\sqrt{r_2}$	0.633	0.633	0.448
Modulus of Rupture f_t = $2750r_1$	1,740	1,740	1,230
$Z_t = r_1 Z_0 = \frac{M_t}{f_t}$	13.50	6.74	9.55
t, from Equation (3) ...	12.5	16.8	19.6

Mr. Lawson, in reply, said that in his opinion there was no serious difference between Dr. Hamilton's treatment of the problem and that of the authors. He and his colleagues had made the tentative assumption that the timber would retain its full strength until charring took place. This was used only to enable an expression to be found having a form likely to fit th

xperimental results. He saw little value in distinguishing between r_1 the section modulus after a time t compared with the initial section modulus and r_2 the load the eam would support after a time t compared with the reaking load of the beam before test.

Dr. Hamilton rightly pointed out the danger in xtrapolating outside the range of the experimental esults, but he thought that a square beam when loaded o a given fraction of its ultimate load would give a onger fire endurance than that of any other beam imilarly loaded. Mr. Lawson agreed that a deep beam vould carry a greater load than a square-sectioned beam ven though they both had the same area of crossection and that if one were concerned with the absolute oading of beams then the square cross-section would ot be the optimum shape.

Since preparing the paper he and his colleagues had nvestigated the fire endurance of timber beams in terms f the working load applied to them. A beam having shape ratio "s" would carry a load $\sqrt{s}$ times as reat as a square-sectioned beam before failure. Thereore, if r_0 is the load ratio for a square-sectioned beam hen the same working load will give a load ratio of $_0/\sqrt{s}$ for a beam having a shape ratio s. The effect of oad carrying capacity on fire endurance for beams of arious shape ratios can be found by replotting Fig. 6 vith $r_0/\sqrt{s}$ as a parameter. This graph is shown in ig. C. For light loads (small values of $r_0/\sqrt{s}$) there is

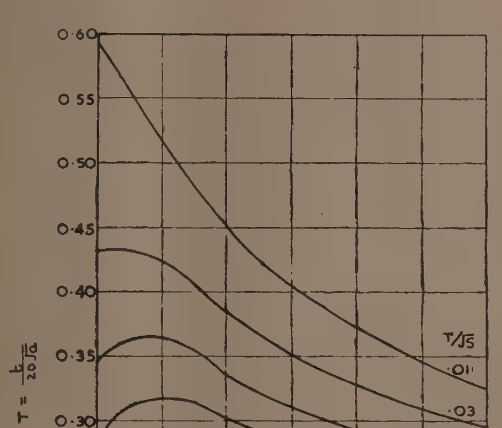


Fig. C.—The fire endurance of timber beams of various sections as a function of the absolute imposed load

some advantage to be gained by using sections where s is about 2, but for beams carrying normal working loads (e.g., $r_0/\sqrt{s} = 0.1$) the effect of shape is small within the practical range $s < 5$. The authors stated that since preparing the paper they had produced a nomogram from which the fire endurance of a beam could be calculated : this is shown in Fig. D.

Mr. T. Bedford (Member) remarked it was somewhat of a platitude to say that theory followed practice but the paper seemed to be a good example of that view and he thought it was remarkable the way the author had produced the theoretical graphs and formulæ to line up so closely with the results of actual tests carried out. In Table 5 they would note that the difference between an estimated endurance of fire and the actual time for collapsing showed only a variation of about ten per cent. in six experiments. He thought that this difference between theory and practice was remarkably small and the authors were to be congratulated on their mathematical analysis.

From the practical point of view they would have liked to have heard a word about the intensity of the heat on those beams. One imagined the heat varied considerably in an actual fire and they must have had some sort of temperature to work from in those tests. Could the authors give them any idea of how that temperature might be related to a temperature which one would expect in an actual fire ?

The other question he would like to ask was concerned with the ideal shape of the beam. It seemed to him that at a meeting like this many would be getting lost in the higher mathematics of the problem, but they understood that the strength of a beam was proportional to the square of its depth or, alternatively, directly proportional to its width. Assuming the case of a 9 in. × 3 in. joist penetration on the soffit which reduced it by one inch reduced the strength by 25 per cent., whereas if there was an inch penetration on the sides of the beam the strength would be reduced by two-thirds (66 per cent.), which indicates that in practice they should give more consideration to the protection of the sides of the beams.

Mr. Ashton, replying to the points raised said that the questions about the fire were very important. They should stress the point that these experiments were carried out under controlled conditions of heat. There was a standard time-temperature curve for fire resistance tests and the portion of the time-temperature curve that had been used for these tests was the early stage when the temperatures were increasing very rapidly and approximated to the kind of rapid development that one got in the early stages of a fire. It was not claimed that the actual temperatures did correspond. Fires varied so much in fact, and the standard time-temperature curve did not correspond to any particular fire, but was a standard of comparison to give a measure of how different structures would behave.

Burn-out tests carried out in experimental buildings in the United States, however, had shown a correlation between the fire-load, i.e. the amount of heat in B.Th.U. per sq. ft. of floor area liberated by combustion of the contents of a room, and the duration of the standard fire test.

The standard temperatures that had been followed in the tests gave a temperature in the furnace of just over 700°C. in ten minutes and of about 850°C. in half an hour. The furnace that was used for the tests was large enough to heat a ceiling area of 10 ft. by 12 ft. and the beams that were made up of two joists with floor board occupied only the central strip of the furnace, the rest being covered with refractory concrete slabs. But the difficulty was in ensuring that the furnace did repeat fairly closely, in every test, the standard temperature. One difficulty with the furnace was that it was very susceptible to atmospheric changes. The flue system was

rather unorthodox, so care was taken to ensure that the flue and the furnace walls were in about the same condition prior to starting every test. Thus, before beginning the series of tests a dummy run was taken on the furnace one day and then followed with tests on successive days. If there was a gap in the testing, the next test started off with a dummy run, and it was found that in this way the furnace was made to reproduce fairly accurately the standard temperatures.

With regard to temperatures actually attained in fires, there was no evidence that the rate of charring of the timber would be very different, even if the joists were exposed to temperatures much higher than those in the furnace. Mr. Bedford's point about the sides of the beam was a good one; it was the width which was important in contributing to fire endurance and protection of the sides of a beam would improve fire endurance considerably.

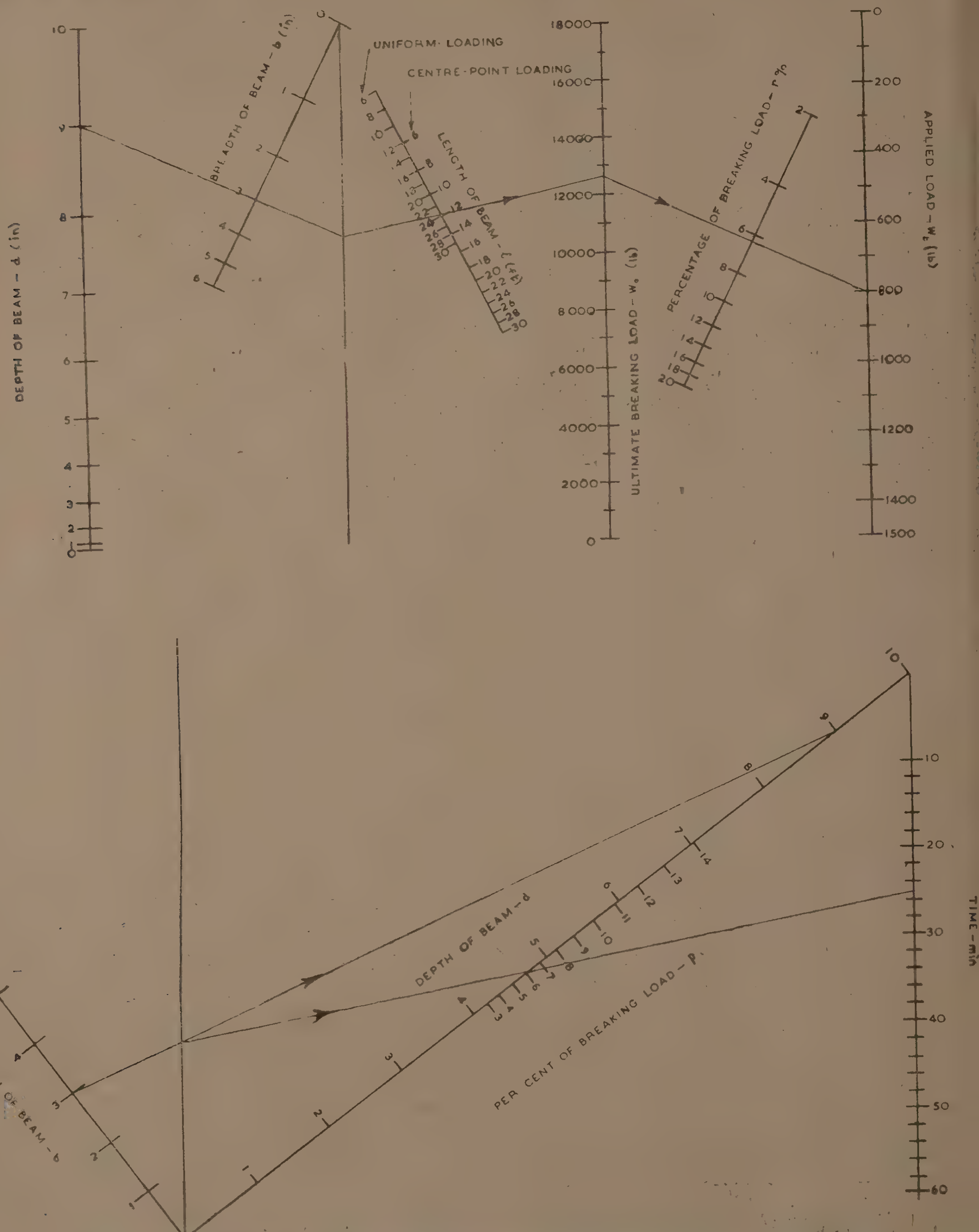


Fig. D. Nomograms for obtaining fire endurance of timber beams (Spruce & Douglas Fir)

They had found, however, with ceilings of practically all types tested, that when the flame broke through, very little of the protection remained in place on the soffits of the joists ; usually it fell away quite quickly and exposed the whole of the sides and soffit of the joists.

Mr. MAURICE HALL wanted to know about the nature of the ceiling and the temperature gradient through such protective coverings. The ceilings quoted in the paper were all good heat insulators so that actual flame penetration might be necessary before the temperature on the concealed side rose to dangerous heights. With metal ceilings such as those used by Woolworths, however, it was conceivable that long before flame penetration had taken place the joists would have been charred by heat conducted through the metal. Between these two extremes of ceiling there might be many—he was thinking of such things as hard boards—where there was doubt as to which would happen first : flame penetration or ignition by conduction. That, he thought, would be fairly important.

Going one stage further in that connection, sprayed applications of vermiculite or asbestos might one of these days become common practice in treating the underside of floors in old buildings of the warehouse type, especially where services made it difficult to put up a ceiling.

To go back to the beginning, therefore, it did seem important that they should have some information on temperature gradients through various kinds of material of various thicknesses and then perhaps the authors could give some more information as to the relative value of various kinds of protection.

Mr. ASHTON thought that although Mr. Hall's question covered a very wide field he could answer it partly as far as the paper was concerned. The Joint Fire Research Organization had been dealing with ceilings of rather low performance and the question of conduction of heat through the ceiling did not arise because the ceilings actually broke up or failed before the heat on the upper surface was sufficient to cause ignition.

Conduction of heat was the determining factor in the vermiculite or asbestos plaster type of ceiling that Mr. Hall mentioned and with these materials Mr. Ashton agreed that ignition of the timber could occur while the ceiling was still in place. There was, however, another factor. Considering the ceiling of metal sheet with a direct application of vermiculite or asbestos plaster which Mr. Hall particularly mentioned, there was a possibility that the plaster adhesion would break down before the temperature on the upper surface was sufficiently high to cause ignition. The Joint Fire Research Organization had not yet had direct experience of that type of ceiling, and the problem of conduction was quite a minor one for the types of ceiling which were commonly used with timber joists.

Mr. B. L. CLARK (Associate-Member), said he would like to enlarge a little on the matter of the merits of hard wood in relation to soft wood, so far as fire resisting properties were concerned, and which had been raised by a previous speaker.

He recalled two particular jobs. One was a large furniture works which had a large store of soft and hard timbers, such as mahogany, teak, oak and the usual pines, etc., and he had been called in after they had suffered a completely destructive fire. The whole of the softwood had been destroyed, but the hardwood had been only partially destroyed, with a considerable quantity being badly charred. The depth of penetration being about a quarter way through the boards of $1\frac{1}{2}$ in. thickness, although similarly stacked.

Recently he had had another case of fire, interposing to say that he had nothing to do with fire assessors or their like.

This particular factory consisting of 15 bays, each 20 ft. span, was entirely destroyed, it having been built entirely of timber of varying types, and covered with bitumen felting, the product inside in the course of manufacture being also of softwood. Some of the timber trusses of oak—it may have been English or some other type—however, remained in quite large pieces.

On the question of Bye-Laws relating to fire-proof doors, the L.C.C. specified two inch teak in preference to any other hardwood ; and they would not allow the use of soft wood, indicating that there was a preference for hard wood where fire resisting qualities were required.

Was there any means of fire-proofing timber by impregnation ? He recalled some method of treating timber with an alkali or something similar, which from records, seemed to retard the ignition point when inflammable gas was given off.

If we still used timber beams where fire resisting qualities were required, then he thought they would last considerably longer if they were clothed in a thin metal skin of say 20 gauge thickness, or some similar medium to reflect the heat and encase the inflammable gases, and thus preventing the complete ignition until the skin had completely disintegrated by melting. This would probably result in the charring of the timber beam, but the strength would be maintained far longer. There was also that proprietary ply-wood faced with metal : did that offer much resistance to charring ?

He would like some further information on this question of hard wood, which interested him, because they had to erect within another building, a room where cellulose spraying would be carried out, and had considered using teak or some similar West African hard wood due to the shortage of steel. It was hoped that this arrangement would meet the insurance company's requirements.

Mr. LAWSON dealing with these points, said that the function of the metal facing was to disperse the heat from the point of application. If heat was applied to any point on a combustible material, the temperature at that point would rise ultimately to a temperature at which the timber would begin to break down and give off combustible gases. This temperature was about 270°C. If there was a pilot flame nearby then combustible gases would be ignited and the surface would begin to flame. The flames from that source would then transfer heat back to the unburnt wood and if the wood could be brought to the temperature at which it began to decompose and give off combustible gases the flame would spread along the wood. If there was no pilot flame, the temperature at which ignition took place would be about 480°C.

If one put a metal facing on the wood then the situation was rather different. If the heat was applied fairly locally, then because the metal was a good conductor, the heat would be dispersed from the point of application and the temperature would not be raised enough, locally, to start the thermal decomposition of the wood. Even if combustible gases were liberated, the metal facing would act as a seal and separate the gases from the igniting source and prevent the wood burning ; it would only begin to char. A metal facing would not be effective in dispersing the heat if the heat were applied to all parts of the wood simultaneously, but while it

remained in position it would seal the gas given off from the wood and to that extent would be beneficial.

Whether impregnating timber would increase its fire resistance depended on what was going to be stored in the room. If it was going to contain a lot of combustible materials then the temperature inside the room would rise to approximately the same temperature irrespective of whether the wood had been impregnated or not, and under those conditions the fire resistance of the beams would not be very different from that of untreated timber. On the other hand, if the wood inside the room was predominantly treated wood, then some benefit would be derived.

With regard to the second part of the question, Mr. Lawson said that there was some evidence on the comparative behaviour under fire conditions of hard woods and soft woods from tests made under the same conditions on boards up to 2 inches thick of different species of timber exposed on one face to the standard furnace heating. The time of flame penetration did not vary significantly for the typical species tested. He would have deduced this result from theoretical considerations since the factor which determined the rate at which that temperature moved inward was the thermal diffusivity, that is the conductivity divided by the product of the density and the specific heat of the wood. As the specific heat of all species of wood was the same, this limited the discussion to the conductivity and the density and they were interdependent. If the density was increased the conductivity of the wood was increased ; if the density was low, the conductivity was low and so, in as far as the quotient of these two remained constant, it would be expected that the rate of charring for the different woods would be similar. For a member such as a joist, exposed to fire on three faces, charring would occur at a certain temperature, probably the temperature at which the wood began to decompose, and if the heating were continued the charring temperature would be reached at points further and further below the surface of the wood until after a time the beam would be charred away.

Mr. C. F. Dennett, Assistant Divisional Officer, Fire Prevention Branch, London Fire Brigade, asked if the authors of the experiments had data for roof members. All the data they had given were for beams with point loading. Had they ever experimented with roof members with a view to arriving at a figure which would ensure that secondary members of the roof would fall first and not, as was so often the case, the principals collapsing first.

Dealing with this query Mr. Ashton said they had not dealt with roof members at all. They were concerned solely with horizontal members as found in timber floors and they did not extend their recommendations to roof members which were a different problem. In the roof one got quite different sets of stress conditions and their predictions would not apply.

A Speaker remarked that his question applied to the actual time of collapse. First he wondered what conclusion the authors came to about the possible effect of the floor itself on the figures that they had given. He had in mind the possible difference between an airtight tongued and grooved floor and an open boarded floor, and specially the time of penetration through the ceiling.

Secondly, the more important matter of the lateral stability of the joists. That was to say, bending the floor. Had they any case where the collapse time was affected or perhaps determined by the beams failing in twist rather than the failure by the technical description they had had that night, by a reduction in direct size ?

They had had the comparable times of the theory and the experiment. They had been told they were in close agreement. Was there any evidence, later obtained, to compare them with actual floor collapse ? Were those figures on the right lines in actual practice ?

Finally had it been possible to investigate the effect of a hose playing on the floor before it collapsed ?

Mr. Ashton said regarding the effect of the floor boards that the Joint Fire Research Organization had realised this factor was quite important, and tongued and grooved board did make a contribution to the endurance of the floor as a whole. The tests on beams which had been described were carried out with plain edge boarding, laid with gaps so as to minimise the stiffening effect of flooring and to achieve a fairly uniform result.

The question of the lateral stability was important, but consideration had to be given to certain factors and all the variables could not be investigated. Experiments had not been conducted on the effect of lateral stability and failure in buckling. It was ensured that the specimens that had been constructed were adequately strutted to avoid failure in twist, which was not typical.

Test results had been obtained with complete floors using both tongued and grooved and plain edge boarding, and there had been an increase in the fire resistance due to the stiffening effect of the tongued and grooved boarding. The stiffening effect could be a variable quantity because in an actual floor its value depended on the workmanship of the boarding. With time, also shrinkage of the boards might occur leading to open joints. The tests were on floors of sound construction. In practice, he considered that tongued and grooved boarding would not present such tight joints as in the test specimens. The difference between tongued and grooved and plain edge board would only appear in the time of collapse of the joists. He did not think that their observations had shown any difference between the time of failure of a ceiling used in conjunction with either type of floor board.

The other question about relation to actual fires was another practical one. Generally, when a fire occurred there was no one present who was detached enough to make observations as to the duration of the floors or the behaviour of the structure, so the only evidence that could be obtained and relied on was a controlled experiment conducted on a full-sized structure, and it had not been possible as yet to carry that out with a timber-joisted floor.

As to the effect of water, the application of a water jet is an integral part of the fire resistance tests of long duration (two hours or more) and was not required for the tests under consideration. He anticipated that the application of water would prevent any further deterioration in the condition of the joists, and if collapse had not occurred when the water was applied then it was assumed that the structure would still be safe.

Another Speaker said he had been rather surprised that Mr. Lawson had suggested that they should stress grade for fire resistance. It was rather difficult to get timber merchants to stress grade at all and he thought the suggestion to do it from the point of view of fire might be impossible.

There was one question he would like to ask. Due to the shortage of steel at the moment, many would want to build in timber and to get a fire resistance in a floor of something more than the half hour that Mr. Lawson had shown. He thought it had been suggested that they should use floors with a concrete pugging to give an hour's fire resistance. He would ask his views on that, its advantages and its disadvantages in using that to bring a building up to the hour classification.

The type of rendering of ceilings on timber floors. He had noticed that gypsum was used in many instances. Was there very much difference between gypsum applied to expanded metal and cement rendering as a finishing coat as far as fire resistance was concerned? Was there any effect on steel due to the use of gypsum? He knew there were two grades and it was probably very appropriate to make certain that they got the proper grade on a job: it was quite easy in the laboratory.

He believed one speaker had spoken of the use of steel and timber, but he did not think anyone in any building would use steel except for roof members and he did not think that very appropriate. Timber would obviously last a greater length of time than any protected steel work. Another speaker had mentioned the fire resistance of stacked soft wood and stacked hard wood. He had been wondering in that case, as Mr. Lawson had pointed out that there was not much difference in the resistance to fire as between hard and soft wood, whether it was not really a question of stacking. In that particular instance had the soft wood been stacked with greater spaces between it than was the case with the hard wood, and so made it easier for the fire to penetrate?

Mr. Lawson said he had to dissent from any recommendation that timber should be fire graded. He would not like to suggest that any more restrictions should be imposed than there were at present. He thought he had said "If one were going to do that . . ." which was quite different.

On the question of designing floors to give a high fire resistance, it would be preferable to spend money on improving the ceiling rather than increasing the size of the beams. If the beam were pugged, that would be one very good way of increasing the effectiveness of the ceiling.

Mr. Ashton, who dealt with the portion of the question relating to the use of plaster on expanded metal, said that a series of experiments had been carried out, with as nearly as possible the same conditions, on different plasters, and almost no difference had been found between the performance of lime/cement plaster and gypsum plaster on metal lath with the normal methods of fixing. The trouble was that usually the full fire protective value of the plaster was not developed because the fixing came away unless the metal lath was given a suspension and fixed to the sides of the joists. The staples of nails which fixed the lath to the soffit came away and let the ceiling down as a whole, and the time at which that occurred was approximately the same for gypsum as for a lime cement.

Special measures, such as side hangers screwed to the sides of the joists or a plaster incorporating vermiculite, were needed to obtain an hour's fire resistance. A lime cement or an ordinary gypsum plaster on a suspended ceiling would be sufficient if a light pugging was used and it had been found that for this purpose mineral wool was very effective, and using this mineral wool, up to two hour's, fire resistance could be obtained with a suspended ceiling of that form.

Mr. M. Chadwick, expressed his thanks for being allowed to attend that meeting and said he had been closely associated with Mr. Clarke and his people at the D.I.S.R. for some time. He was very interested in the question of the attempt to protect timber from fire.

Mr. Lawson, he understood, had expressed some doubt as to whether one could satisfactorily protect timber by a metal casing. He was inclined to agree whole-heartedly with him because of the problem of conductivity which, he thought, had probably escaped notice. On that side of the question most of the Fire Service approved the splendid protection afforded by the two inch teak door and they probably regarded that as a better means of preventing fire spread than even steel.

The point he wished to raise was the conductivity of metal itself. Those in the services who had been dealing with fires on ships or in steel houses could appreciate the great problems encountered due to conductivity through metals.

In the case of ships, beams and bulkheads were all steel and one would consider them fairly fire proof, but the facts of the case showed that this was not so. Instead, the heat conductivity of the steel tended to set fire to the paint work and similar combustible materials on the other side of the metal and hence produced a very rapid spread of fire.

He said that Mr. Lawson had not amplified this point, but there was something to be said for the use of good hard wood, inasmuch as it did not have the failings of steel, such as collapse and expansion, and above all by the fact that it did not conduct heat to other parts so rapidly as steel. The problem in steel houses which we had come up against was the terrific spread of fire and heat through the steel frames and steel sheets of the houses. They had to do something to insulate against heat and stop it travelling through the steel and it was doubtful if they could protect timber satisfactorily with light steel facings.

Mr. Lawson in reply said he thought there was very little to add to what he had said previously with regard to the protection of wood by steel facings. This certainly did assist in preventing ignition taking place. Once the fire had become well established, so that the metal was heated all over, then it could not act as a disperser of the heat: the heat was rapidly conducted through to the wood and the wood would char.

With regard to metal construction, steel itself was an unsatisfactory material in fire. If one had a steel column inside a building which was unprotected, then it would fail after a matter of ten ninutes. Wherever steel was employed some sort of insulation had to be used round it to protect it from heat, otherwise the steel would soon reach a temperature at which it lost strength. One had always to encase steel in some form of poor thermal conductor in order that the temperature should be kept down, otherwise the steel would fail quickly.

Mr. D. N. Mitchell (Member), said that he did not think they need be concerned much at present whether it was a good thing to case floor joists in thin steel plate, since steel was in such short supply. Had any thought been given to alternative materials which might be used? Had the authors, or any of their associates, carried out similar tests on prestressed concrete joists, and if so what was the fire endurance of these?

Mr. Lawson mentioned that the covering of joists with metal would be mainly effective in preventing ignition. It was much better to cover the timber with something that was not a good conductor of heat. The thermal insulation applied to wood stopped the heat reaching the wood and therefore prevented ignition.

With regard to prestressed concrete, the Joint Fire Research Organization had carried out some tests to find out under what circumstances prestressed concrete did completely fail in a fire. The aim was, in the first place, to find the minimum kind of protection that would be necessary and also to find what particular conditions and degree of restraint would favour spalling.

Closing the meeting, the Chairman thanked the Authors for the way in which they had answered the questions raised in the discussion.

Institution Notices and Proceedings

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, October 9th, 1952

Presidential Address, by Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E., at 6 p.m.

Thursday, October 23rd, 1952

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. A. J. Harris will give a paper on "Prestressed Concrete Hangars at London Airport."

JANUARY EXAMINATIONS

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on January 6th and 7th, 1953 (Graduateship), and January 8th and 9th (Associate-Membership).

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer:

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in the first category provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

The Colleges which have informed the Institution that courses in Structural Engineering are available are:

- Belfast College of Technology.
- Birmingham College of Technology.
- Bolton Municipal Technical College.
- Bradford Technical College.
- Derby Technical College.
- Dudley and Staffordshire Technical College.
- Glasgow Royal Technical College.
- City of Liverpool College of Building.
- L.C.C. Brixton School of Building, S.W.4.
- L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
- Manchester College of Technology.
- Middlesbrough, Constantine Technical College.
- Salford Royal Technical College.
- South-West Essex Technical College, Walthamstow, E.17.
- Stockport College for Further Education.
- Willesden Technical College, N.W.10.

Colleges in the second category provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete. The colleges which have informed the Institution that such courses are available are:—

- Brighton Technical College.
- Cardiff Technical College.
- Huddersfield Technical College.
- Leeds College of Technology.
- London, Battersea Polytechnic, S.W.11.
- London, Northampton Polytechnic, E.C.1.
- L.C.C. Westminster Technical College, S.W.1.
- Plymouth and Devonport Technical College.
- Preston, Harris Institute.
- Wigan Mining and Technical College.
- Woolwich Polytechnic, S.E.18.

Students attending colleges in the first category are advised to take the organised courses in Structural Engineering. Students of Graduate Membership standard will usually be allowed to select subjects from courses provided by colleges in the second category.

RESEARCH AWARDS

The Council have instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following:—

(a) investigations of an experimental or analytical character;

(b) studies of historical or statistical records ;

(c) improvements in principles or methods of construction ;

(d) research into methods of structural engineering ad building, the nature and use of plant and the rganisation of engineering work ;

(e) any related or combined studies which are deemed y the Literature Committee to be of a research haracter.

In cases where the research work described in the aper was not the work of one individual, the names of ll the collaborators should be given in the paper.

Awards may take any or all of the following forms :— research medal ; a diploma ; a money prize.

Application for consideration for a research award nust be made to the Secretary of the Institution, and in reparing papers for reproduction in the Journal, authors nust comply with the conditions laid down for all such ontributions. Particulars of these conditions may be btained from the Secretary.

In judging research papers, the following factors will e considered :—

(a) the nature of the subject and its conclusions ;

(b) the value of the paper in advancing the science nd art of structural engineering ;

(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1951, and September, 1952, is October 31st, 1952.

LONDON GRADUATES' AND STUDENTS' SECTION

A visit has been arranged to Battersea Power Station on Saturday, September 6th, at 10 a.m. The party will meet at the Kirtling Street entrance at 9.45 a.m. (Kirtling Street is a turning off Battersea Park Road). As the party is limited in number, members wishing to attend are asked to inform the Honorary Secretary as soon as possible, and unless they hear to the contrary, this notification will secure a place in the party.

Hon. Secretary : C. Allen Brown, 43, Coolgardie Avenue, Highams Park, London, E.4.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The constitution of the Branch Committee for the Session 1952-53 is as follows :—

Chairman : W. Bates (Member).

Vice Chairman : Professor J. A. L. Matheson, M.B.E. (Member).

Immediate Past-Chairman : R. Gray (Member).

Hon. Secretary : A. S. Sinclair (Associate-Member), 28, Kenwood Road, Stretford, Lancs.

Hon. Assistant Secretary : M. D. Woods (Graduate).

Hon. Auditors : F. Walkden (Associate-Member), K. Norrey (Associate-Member).

Committee : W. D. Blades (Member), A. V. Booth (Member) (Past Chairman), F. C. Brookhouse (Past Chairman), S. Gleaves (Member), J. B. G. Martin (Member), P. Mather (Member) (Past Chairman), J. H. Morris (Member), G. A. Davis (Associate-Member), G. Greenlees (Associate-Member), D. D. Mathews (Associate-Member), K. Norrey (Associate-Member), A. E. Wright (Associate-Member).

The opening meeting of the Session will be held on October 7th, and will be attended by the President and Secretary of the Institution.

MIDLAND COUNTIES BRANCH

The constitution of the Branch Committee for the Session 1952-53 is as follows :—

Chairman : H. J. Morris, M.B.E. (Member).

Vice-Chairmen : G. E. Marsden (Member), W. Phillips (Member).

Hon. Treasurer : H. Ferrington (Member) (Past Chairman).

Hon. Auditors : L. P. B. Arthur (Member), E. Jones (Associate).

Hon. Secretary : L. A. Firminger (Associate-Member), 656, Chester Road, Erdington, Birmingham, 23.

Hon. Assistant Secretary : J. C. Billington (Associate-Member).

Acting Hon. Assistant Secretary, Derby District : O. W. Jones (Member).

Committee : J. W. H. Chattaway (Member), W. D. Christie (Member), A. T. Clark (Member), E. R. Deeley (Associate-Member), B. D. Evans (Member), R. J. Fowler (Member), L. J. Griffiths (Associate-Member), G. Kilner (Member), W. J. N. Mayo (Associate-Member), F. B. Watson (Member).

Ex-officio Members of Committee : Member of Council. B. C. Britton (Associate). All Subscribing Past Chairman (who are members of the Branch).

The Branch Annual Dinner will be held at the Botanical Gardens, Birmingham, on Saturday, October 11th.

The opening meeting of the Session will take place on Friday, October 24th. The President and the Secretary of the Institution will attend on both occasions.

GRADUATES AND STUDENTS' SECTION

The Committee for the Session 1952-53 is as follows :—

Chairman : S. M. Cooper (Associate-Member).

Vice-Chairman : J. E. Taylor (Graduate).

Hon. Secretary : F. G. Fletcher (Student), 60, Brean Avenue, South Yardley, Birmingham, 26.

Hon. Assistant Secretary : J. S. Allen, A.M.I.C.E. (Graduate).

Committee : C. B. Brewington, B.Sc. (Graduate), P. J. Clark (Student), M. H. Evans, B.Sc. (Graduate).

The opening meeting of the Session will take place on Thursday, October 30th.

NORTHERN COUNTIES BRANCH

The opening meetings of the Branch will be held at Middlesbrough on Tuesday, October 14th, and at Newcastle on Wednesday, October 15th, when the Chairman's Address will be given by Mr. A. V. Buttress. The meetings will be attended by the President and the Secretary of the Institution.

Hon. Secretary : Ian MacGregor, M.I.Struct.E., Messrs. H. Pickup, Ltd., Roscoe Street, Scarborough.

NORTHERN IRELAND BRANCH

The constitution of the Branch Committee for the Session 1952-53 is as follows :—

Chairman : M. C. Gillies (Member).

Vice-Chairman : Major M. T. Shaw (Member).

Immediate Past Chairman : Howard Harding (Member).

Hon. Auditors : W. S. Benton (Retired Member). W. A. Plester (Associate-Member).

Hon. Secretary : S. G. Duckworth (Member), "Lisleen" 13, Finaghy Road North, Belfast.

Hon. Assistant Secretary : J. M. C. Tyack (Associate-Member).

Committee : L. Clements (Associate-Member), R. Montgomery (Associate-Member), T. A. N. Prescott (Associate-Member), A. H. K. Roberts (Member), R. J. N. Sweetnam (Graduate).

The opening meeting of the Session will be held at the College of Technology, Belfast, on Tuesday, October 7th, at 6.45 p.m., when the Chairman's Address will be given by Mr. M. C. Gillies (Member). The meeting will be preceded by tea at 6 p.m. at the Overseas League Premises, Wellington Place, Belfast.

SCOTTISH BRANCH

The opening meeting of the Session will be held on Monday, October 27th, and the Annual Dinner on October 28th. The President and the Secretary of the Institution will attend.

Hon. Secretary : D. G. Drummond, B.Sc., M.I.Struct.E. A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

The Annual General Meeting of the South-Western Counties Branch was held at Plymouth on Friday, May 16th, when the following Honorary Officers and Committee members were elected for the Session 1952-53 :—

Chairman : L. F. Vanstone (Member).

Vice-Chairman : F. J. Powell, M.B.E. (Associate-Member).

Hon. Secretary and Treasurer : E. W. Howells (Member), c/o Messrs. T. Harding & Sons, Ltd., 10-12, Market Street, Torquay, Devon.

Assistant Hon. Secretary : C. J. Woodrow (Graduate).

Hon. Auditors : H. J. Scoles (Member), and J. C. Peters (Associate).

Committee : Col. F. J. Dean (Associate-Member), A. C. H. Harris (Associate-Member), F. M. Upson (Associate), W. C. Tyler (Associate-Member), H. Toft (Associate-Member), H. W. G. Miller (Graduate), Colonel R. Hazzledine, O.B.E. (Member), F. W. Potter (Associate-Member).

The opening meeting of the Session will be held at the Duke of Cornwall Hotel, Millbay, Plymouth, on Wednesday, November 5th, and will be attended by the President and the Secretary of the Institution.

WESTERN COUNTIES BRANCH

The constitution of the Branch Committee for the Session 1952-53 is as follows :—

Chairman : E. N. Underwood (Member).

Vice-Chairman : N. G. T. Ball (Member).

Hon. Vice-Chairman : Ewart S. Andrews (Past President).

Past Chairmen : Gower B. R. Pimm (Past President), Professor J. F. Baker, O.B.E. (Member), C. H. Williams (Member), C. J. D. Boxall (Member), P. C. Girdlestone (Member), Professor A. G. Pugsley, O.B.E. (Member).

Hon. Secretary : E. Hughes (Associate-Member), 39, Effingham Road, St. Andrew's Park, Bristol, 6.

Hon. Treasurer : E. K. Fennell (Associate-Member).

Hon. Auditors : F. A. Long (Member), and J. M. Rome (Member).

Committee : R. H. Barnett (Member), F. G. Clarke (Associate-Member), J. W. Lorraine (Member), G. F. Poppleton (Associate-Member), C. E. Saunders (Member), Lt. Col. E. Ward, T.D. (Associate-Member), R. L. Bourqui (Associate-Member), G. C. Mander, M.B.E. (Associate-Member).

The opening meeting of the Session will be held in the University of Bristol Geology Lecture Theatre on October 10th, at 6 p.m., when the Chairman's Address will be given by Mr. E. N. Underwood (Member). The President and the Secretary of the Institution will attend the meeting which will be preceded by tea at 5 p.m.

WALES AND MONMOUTHSHIRE BRANCH

At the Annual General Meeting of the Wales and Monmouthshire Branch, held at the South Wales Institute of Engineers, Cardiff, on Tuesday May 6th, 1952, the following Honorary Officers and Committee were elected :—

Chairman : Colonel R. D. Heseltine, T.D., D.L. (Member).

Senior Vice-Chairman : Professor W. N. Thomas,* C.B.E. (Member).

Junior Vice-Chairman : T. B. Richard* (Member).

Honorary Secretary : G. R. Brueton (Associate-Member). 2, Celtic Road, Gabalfa, Cardiff.

Asst. Honorary Secretary for North Wales : S. C. Brown (Associate-Member).

Honorary Auditors : H. G. Hope and G. W. Spooner (Associate-Member).

Committee : G. H. Hodgson* (Member), W. A. Evans* (Member), A. G. Thompson* (Member) (Past Chairman), D. Manolopoulos* (Member), Dr. A. A. Fordham* (Member), A. V. Hooker (Associate-Member), E. O. Jones (Member), J. E. Jenkins (Associate-Member), H. V. Morris (Associate-Member), J. L. Bannister (Associate-Member), W. D. Hollyman (Associate-Member), K. J. Stewart (Associate-Member), E. R. Steward (Associate-Member).

The opening meeting of the Session will be held at Cardiff on Friday, October 17th, when the Chairman's Address will be given by Colonel R. D. Heseltine (Member). The meeting will be attended by the President and the Secretary of the Institution.

YORKSHIRE BRANCH

The opening meeting of the Session will be held on Thursday, October 30th, and will be attended by the President and the Secretary of the Institution.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

At the Annual General Meeting held on May 28th, the following Honorary Officers and Committee Members were elected :—

Chairman : Dr. A. J. Ockleston (Member).

Vice-Chairman : F. F. Binswanger (Member).

Hon. Secretary : A. E. Tait (Associate-Member).

Committee : D. R. Ryder* (Member), C. A. Pringle (Member), C. A. Rigby (Member), J. G. Hay (Member), G. M. Frost (Member), T. Breslin (Associate-Member), D. D. Thorp (Associate-Member), E. Kretzschmar (Associate-Member).

Hon. Secretary, Natal : E. G. Bennett (Associate-Member).

Hon. Secretary, Cape : R. Stubbs (Associate-Member).

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

**Past Chairman*

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

CORRESPONDENCE

The Institution, whilst being at all times pleased to open its columns to correspondence, cannot accept any responsibility for the opinions expressed.

ANALYSIS OF CONTINUOUS RIDGED PORTAL FRAMES

To the Editor of THE STRUCTURAL ENGINEER.

Sir,

The paper by Mr. Markland* is of considerable interest in giving the application of relaxation methods to a multi-bay gabled framework.

The application to simpler problems is very clearly explained but I believe the paper would be even more valuable if the solution of a general problem had been given.

A practical problem will invariably include an unsymmetrical load on the rafters, and quite a large proportion of practical designs are of north light or other unsymmetrical form.

In these latter cases the considerable degree of simplification present in Mr. Markland's solution is lost. Not only is the calculation of the fixed end constraints much more difficult, but so is the calculation of the operations to relax them. There is also the question of how best to obtain the value of the bending moment at the ridge joints.

Members present at the January meeting of the Institution will recall that the complete solution of a five bay unsymmetrically loaded north light framework was presented. I believe that the full power and value of the relaxation approach is only seen in such more difficult general cases.

Manchester,
16th May, 1952.

Yours faithfully,
A. BOLTON.

*The Structural Engineer, Vol. XXX No. 5, pp. 101-108, May 1952.

Book Reviews

Beton Precontraint, by Y. Guyon. Preface by E. Freyssinet. (Paris : Eyrolles, 1951.) Pp. 728, 10 in. × 7 in., 503 Figs. 4,500 Fr.

As the author points out in the preface, prestressing is a general principle which may be employed by different means in a great variety of cases. To write a complete treatise on the subject seems to be an impossible task. The purpose of the author is to develop general principles by analysing a few applications in detail and to describe the methods of execution which may be adopted for other applications. The present book, comprising 700 pages, is the first volume of a comprehensive work on the subject.

Apart from short references to the systems Magnel, Baur-Leonhardt and Chalos, the author has confined himself to the methods developed by Freyssinet. He considers that there are no other inventors in the field of prestressed concrete. " Il y a la PRECONTRAINTE et la Précontrainte c'est FREYSSINET " !

The book is divided into three parts, which contain altogether 18 chapters. In the first part the principles of prestressing are explained and illustrated by examples. Full details are given of various sizes of Freyssinet cables and cones which may be considered as standardised. Data on the creep of concrete and of various types of steel and on the loss of prestress due to friction based on experimental results are included. The chapter on fire resistance contains particulars of tests on three different types of floors carried out at the Building Research Station. It is rather surprising to find details of these important tests first published in a French book.

Of great interest is the analysis of stresses in the neighbourhood of anchorages (in the case of post-tensioned beams) and near the ends of pretensioned beams where the prestress is gradually transferred to the concrete by bond. For both cases a simple method is developed for determining the reinforcement which is necessary to prevent cracking at the ends of the beams.

The second part deals with the " elastic " design of statically determinate straight beams of constant and variable depth. The term " elastic " design means that a straight line distribution is assumed and only uncracked sections are considered. The method adopted is one by trial and error and the section is deemed to be satisfactory if under the limiting conditions (i.e., dead load only and dead + live load) certain permissible concrete stresses are not exceeded. It is realised, however, that this in itself does not guarantee an adequate factor of safety and the last two chapters are devoted to ultimate load conditions, for which a cracked section with a plastic distribution of the concrete stresses is assumed. The author points out that the structure must have an adequate factor of safety against failure and that structures where the tensile resistance of the steel exceeds the compressive resistance of the concrete (known in this country as " over-reinforced ") are disadvantageous. He emphasises that such sections should be used with caution since their safety depends on the quality of the concrete, which is erratic.

The design of post-tensioned beams with positive anchorages where the prestressing force is counteracted by the dead weight of the beam, and the design of pre-tensioned beams with bonded wires where the favourable effect of the dead weight cannot be taken into account, are treated separately. For post-tensioned beams the notion of a " critical span " is introduced, which depends on the live to dead load ratio and the permissible concrete stress under the design load. If the span does not exceed this limit, the carrying of the dead weight " does not cost anything," since it only affects the eccentricity of the cables. In the case of bonded wires great importance is attached to their " even " distribution in the section. It is demonstrated by several examples how this can be achieved either analytically or graphically.

In the third part three large-scale tests on post-tensioned beams and a great number of tests on smaller pre-tensioned beams with bonded wires are described and analysed. Tests in shear are also included. The results are summed up in the last two chapters with a view to establishing general principles regarding the security of prestressed beams.

It may be seen from this short survey that the book contains a vast amount of material hitherto unpublished, very useful to designers of prestressed concrete structures. Nevertheless, there are several essential points about which the reader must be warned. One is the principle of distributing the wires over the whole cross-section in pre-tensioned beams. The author takes it as granted that this is necessary, in contrast to post-tensioned beams. Actually there is no reason for such a distribution. Even if no tensile stresses are allowed

under the effect of prestressing, which is an arguable requirement, this can be achieved in a more efficient way (i.e., with less steel) than in M. Guyon's examples, as proved by many jobs carried out in this country.

The amount of steel to be provided is governed by ultimate load conditions. According to M. Guyon, and with notations used in this country $M_{ult} = .9xd \times A_S \; xt_{ult}$.

Here d denotes the distance of the centre of gravity of the steel from the compressed edge. In the case of a rectangular cross section with evenly distributed reinforcement and no tension at the top, $d = \frac{2D}{3}$.

The equation is given both for pre-tensioned and post-tensioned beams, disregarding the question of bond for the latter. It is based on the assumption that bonded wires near the neutral axis develop their ultimate strength just the same as wires at the bottom. This assumption is apparently justified by test results, because in beams where the reinforcement is concentrated as far as possible from the compressed edge, the calculated tensile stresses at failure generally exceed the ultimate strength of the wires as obtained from tensile tests in the air.

Whilst the adoption of an even distribution of the wires has no other but economic disadvantages, it may be dangerous to rely on M. Guyon's equation in the case of non-bonded or inefficiently grouted cables or in cases where losses of prestress due to friction occur.

Even for pretensioned beams the coefficient of .9 is not always applicable. It may be more or substantially less, depending on the relative strength of the tensile and compressive zones, but for post-tensioned beams it may lead to a gross over-estimation of the ultimate load.

Apart from many experiments carried out in this country which confirm this statement, M. Guyon's second example of large-scale tests on post-tensioned beam shows that his formula is not applicable indiscriminately to post-tensioned beams. The maximum bending moment on this beam was 93.2 tm. Although the beam did not fail completely some damage to the concrete was observed. The load-deflexion diagram indicates that a further increase of the load would have hardly been possible. Using the formula quoted

$$M_{ult} = .9 \times (.708—.059) \times 1410 \cdot \frac{140}{1000} = 115.3tm,$$

i.e., 23.5 per cent. more than obtained in the test. (This comparison is not given in the book.)

In his third example of a post-tensioned beam, M. Guyon himself shows a deficiency of 6 per cent. against the theoretical value, although a simplified formula should be on the safe side. On the other hand, in the first example the test load exceeded by 20 per cent. the theoretical load at which the steel should have failed. If allowance is made for the contribution of the mild steel in the bottom flange, the excess is still 12.5 per cent. The explanation of this surplus load-bearing capacity given by the author is not at all convincing and one has to doubt the reliability of the measurement of the load which was applied by four hydraulic presses.

In estimating the ultimate load of prestressed beams it is essential to realise the difference between bonded and not bonded wires. Even if the applied prestress is as high as 80 per cent. of the tensile strength of the wires (as seems to be standard practice in France) one cannot expect to reach the ultimate strength of the wires if the bond is not efficient.

It is a pity that the book is marred by many printers' errors and that it does not contain an alphabetical index which would greatly facilitate its use. However, in spite of its shortcomings it is a very useful book, which should be carefully and critically studied by every engineer engaged on the design of prestressed concrete.

K. H.-K.

The Displacement Method of Frame Analysis, by G. P. Manning. (London: Concrete Publications.), 1952. 128 pp. 9 in. × 6 in. 9s.

All members of the Institution who are also readers of "Concrete and Constructional Engineering" will already have had preliminary notice of this method of analysis, by reason of the author's recent contributions in that journal, and they will welcome publication of his system in the form of a book such as this—well-produced, convenient in size, and moderate in price. For those who have not had this introduction to Mr. Manning's method, and would like to place it in the genealogical tree which issues from the slope-deflection equations, it may be described as next-of-kin to that recently expounded by Mr. A. Bolton at an ordinary meeting in the Session 1951-52, and published in the January, 1952, issue of this Journal.

The author takes the displacement of the joints in rotation as the variables in his equations, together with the lateral displacements if there is sidesway. The equations are derived from a system of deformation patterns in which each displacement is considered separately. For the most part the problems worked out involve only three or four equations, so that there is never any serious difficulty in obtaining solutions. If the matter had been treated as an academic exercise one might well have asked what should be done when the number of equations is large, as it may well be in theory. The answer is that the author is dealing throughout with practical design problems in which "only those members near the loaded member are appreciably affected," or for which "die-away" factors can be readily computed or estimated. This is sound commonsense, because the hypothetical problems conjured up to exhibit methods of dealing with the Hardy Cross nightmare of slow convergence have usually very little connection with good engineering design. A commendable feature, not often met in books of this weight, is the treatment of members of non-constant section in which full explanations are well supported by design charts.

E. H. B.

Resistance Strain Gauges: Their Construction and Use, by J. Yarnell, B.Sc., A.Inst.P. (London: Electronic Engineering, 1951.) 8½ in. × 5½ in. 128 pp. 12s. 6d.

This book aims at giving a critical introduction to the subject of the use of wire resistance strain gauges to experimental engineers and designers and to young workers beginning their training.

The author deals in a practical manner with the construction and application of resistance strain gauges and with the most commonly used circuits and apparatus. A chapter is included dealing with strain-sensitive lacquer, and a short chapter on the theory of stress and strain in a surface introduces a comprehensive treatment of the theory and measurement of two-dimensional strain analysis.

The President 1952-1953

Mr. Ernest Granter, ho will be installed as resident of the Institution ı October 9th, is a Civil ngineer with the London ounty Council.

His early life was spent Southampton. Born at ıe end of the last century, e was of the first generaon to be suddenly thrust ıto the grim realities of ar while still of school ge. In 1916, when he 'as only 17, he joined the loyal Flying Corps, and e served in the R.F.C. nd the Royal Air Force or the next three years.

On demobilisation in 919, he studied Civil and lechanical Engineering for hree years at University ollege of Southampton, here he took the degree f B.Sc.(Eng.) of London Jniversity. Following his raduation, he became a upil of Mr. F. E. Wentvorth-Sheilds (a Founder Member and Past-President f the Institution) at outhampton Docks for raining as a Dock and Iarbour Engineer.

Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E.

He is at present a Major in the Engineer and Railway Staff Corps, R.E. (T.A.).

The President has been connected with the Institution of Structural Engineers for the past thirty years, having been elected to Graduateship in 1922. He became an Associate-Member in 1924 and a full Member in 1934. His active participation in the scientific work of the Institution dates from 1928, when he was appointed Honorary Secretary of the Science Sectional Committee on Brickwork and Masonry. In 1935, he became a member of the Literature Committee, on which he served for several years. He has since served on most of the main Committees of the Institution and has been Chairman of the following : the Finance and General Purposes Committee, Membership Committee, Education Committee. He has also served on many of the Sub-Committeess and Panels of the Science Committee, including the Soil Pressure Sub-Committee.

In 1924, Mr. Granter was appointed Engineer at Becontree Housing Estate for the construction of roads, sewers and other engineering works required in connection with the building of over 25,000 houses giving accommodation for a population of about 115,000. This was, at the time, the largest municipal housing estate in the world. After more than two years at Becontree, Mr. Granter was transferred to County Hall, where he was engaged in the design of railway bridges, Lambeth Bridge, and the enlargement of the Tramway Subway from Victoria Embankment to Southampton Row to accommodate double-decked tramcars. From 1929 he spent four years as Assistant Resident Engineer and subsequently as Resident Engineer on the construction of Lambeth Bridge. Since 1933, his work has included the design and construction of many major improvements in the County of London, including Thames bridges, subways, tunnels, viaducts, road and railway bridges, roads and river walls.

During the war of 1939-1945, Mr. Granter was seconded to the Ministry of Home Security for part-time duties at Regional Headquarters, London Civil Defence Region for the co-ordination of repairs to roads, bridges, sewers, river defence works and public utility services damaged by enemy action. During this period, he was also concerned with the design and construction of A.R.P. works, such as emergency Thames bridges and supplementary water supplies for fire fighting.

Mr. Granter's membership of the Council began in 1931, as an Associate-Member ; in 1939 he was elected to the Council as a Member. He held the office of Honorary Secretary in 1943-44, Honorary Treasurer in 1945-46 and has been a Vice-President since 1946. He was a member of the Study Committee on Reinforced Concrete, which the Institution convened at the request of the Minister of Works in 1942 to study post-war problems in the building industry. Mr. Granter has also taken an active part in the work of the Codes of Practice Committee and is Vice-Chairman of the Earth Retaining Structures Committee, which has recently completed the "Civil Engineering Code of Practice No. 2—Earth Retaining Structures."

The Institution is to be congratulated on having as President for the coming Session one who has such a complete knowledge of every aspect of its work. In view of the ever-widening scope of the Institution's activities, this knowledge will be of the utmost value in directing its affairs. During twenty years, Mr. Granter has given continuous service to the Institution in almost all its activities and in that period he has become known to a large number of members, who on acclaiming their President for the Session will not only welcome him as a colleague but also as a friend. Those who will have the privilege of serving under him look forward to a pleasurable period of activity and expansion in the field of structural engineering.

Hangars at London Airport Design of Large Span Prestressed Concrete Beams*

By A. J. Harris, B.Sc.(Eng.), A.M.I.C.E.

Summary

The paper describes the use of prestressed concrete in the primary and secondary beams of a block of 10 hangars at London Airport. The primary beams were 150 ft. clear span and were cast in situ with precast diaphragms; the secondary beams were 110 ft. clear span and were precast in segments 7 ft. 2 in. long.

(I) Introduction

In the spring of 1950, the Air Ministry, acting on behalf of British European Airways, invited contractors to prepare schemes and offer tenders for a group of 10 hangars at London Airport. Contractors were to satisfy certain general requirements but were left at liberty to choose their own structural material and form; schemes were in fact submitted in steelwork, reinforced concrete and prestressed concrete. The contract was awarded to Messrs. Holland & Hannen and Cubitts, Ltd., whose scheme contained post-tensioned prestressed beams for the main hangars and pre-tensioned prestressed beams for the workshops, store-rooms, etc., these hangars thereby constituting one of the largest prestressed concrete buildings yet erected. The author's firm, the Prestressed Concrete Co., Ltd., was invited by Messrs. Holland & Hannen and Cubitts, whose Consulting Structural Engineer was Mr. A. E. Beer, A.C.G.I., M.I.Struct.E., to collaborate with them on the design of the primary and secondary hangar beams, and this design work forms the subject of this paper.

Messrs. Scott and Wilson were the Consulting Engineers on behalf of the authorities; the scheme was submitted to their examination and the work carried out under their supervision.

The sub-contractors for the precast segments of the secondary beams were Messrs. Girling's Ferro-Concrete Co., Ltd.

Plate 1. General view of South Facade during construction

The pretensioned purlins in the roof deck were designed and manufactured by Messrs. Concrete Development Co., Ltd.

(II) The Problem

The ensemble consists of two rows of five hangars each facing outwards; between them are two separate rows of workshops, each communicating with its row of hangars and separated by an access road. At the end of the building is a stores shed passing right across the whole width of the hangars and workshops.

Each hangar is 110 ft. deep and 180 ft. long; the total length of 900 ft. is completely free of obstruction and gantry cranes are able to move from one end to the other. The back wall of the hangar is pierced by small doors giving access to the workshops; in the front

*Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 23rd, 1952, at 6 p.m.

an opening of 150 ft. clear span in which are fixed savian doors.

The clear height under the doors was required to be o ft. ; the clear height under the roof was 43 ft. min., he extra being necessary to accommodate the gantries rithout the hook level being lower than the door soffit. he height to ceiling of the workshops was 32 ft.

Each hangar was required to be completely separate tructurally from its neighbours, to enable hangars to e occupied and put into service as they were completed. ach hangar roof thus took on the aspect of a rectangle upported along one side and at two corners.

Initially the scheme provided tracks for two 55 ft. ranes bearing on a common beam at the middle of the 10 ft. span passing right through the whole length of he row of hangars. This was later amended to a rovision for cranes spanning the whole 110 ft. span.

A total of 45 per cent. of roof area was to be glazed.

a secondary one spanning between the door and the wall at the back.

The beams constituting the primary and secondary structures will be considered separately.

Primary Beams

These beams were subjected to loading caused by dead load and live load on the main roof, live load of the cranes, and dead load and live load on the canopy. Some or all of these loads could necessarily be applied off centre, the roof and canopy live loads, moreover, could be either positive or negative. Horizontal wind loads had also to be considered and, unless the rear wall were made much stiffer than would otherwise be necessary, these could only be carried by the primary beam. This beam has thus to carry both torsion and horizontal bending as well as vertical bending. Two solutions were considered :

Plate 2. Internal view during construction

(III) Principles of a Solution

The major factors in the choice of structural form were the following :

(1) The loads on the centre crane rail could reach a maximum value of two point loads of 22.5T each at 16 ft. centres. These loads gave the roof the character more of a highway bridge than a simple roof, and it was thought advisable to employ solid web beams supporting lightweight decking. Other possibilities suggested themselves (shell construction as at Karachi, bowstring girders, etc.), but a solid web girder seemed the only form whose shear and fatigue strength could be relied upon with sufficient certitude under such heavy concentrated loads.

(2) Each hangar had to be structurally separate from its neighbours. Any thought of making either primary or secondary beams continuous over several hangars had therefore to be rejected and the solution was adopted of erecting a primary structure over the door, supporting

(1) Making the beam flexible in torsion and fixing it to the ends of the secondary beams, thus translating all torsional effects into bending of these beams, the horizontal loads being carried by the back wall acting as a cantilever. This solution seemed highly complicated in erection ; it had the advantage of saving headroom, but there was in fact no difficulty in fitting the alternative scheme into the vertical dimensions specified.

(2) Making the beam stiff and thereby supporting on it all horizontal and torsional effects. The beam being a large one in any case, no difficulty was found in designing it for this purpose and erection was greatly simplified, since the construction of this rigid " spine " could proceed without awaiting the secondary beams.

Consideration was given to making the door beam rigid with its supports, thus constituting a portal. This was rejected more for reasons of prudence, perhaps, than for precise technical reasons. It was felt that while certain savings of material would result, they might

well be outweighed by added erection complications. Technical objections did exist, however; notable amongst them was the fact that if the beam were to fill all the space between door soffit and roof-beam soffit, for which a minimum dimension of 12 ft. had been specified, it would give an extremely stiff portal frame, one in which temperature effects would be troublesome. Furthermore, the minimum clear height specified was very little less than that required by an economically designed simply supported beam and the extra headroom could easily be made up by notching the ends of the secondary beams; in consequence no significant saving of dead weight could be made by employing a portal. The only economy therefore would be that of steel in the beam, against which would be offset the steel required in the legs.

occupying the full depth between roof level and the hangar clearance level was designed. The distribution effect was greatly reduced by the interruption of this transverse stiffening beam every 180 ft. but the effect was nevertheless considerable.

The beam was designed as a semi-infinite beam on an elastic foundation; the stiffness ratios were such that the secondary beams constituted a virtually continuous foundation, and the effect of discontinuity was appreciable, but small. An augmented secondary beam was provided at each end of the 180 ft. to carry the concentration of load at the extremities. The calculations were based on the theory set out in "Beams on Elastic Foundations," by Hetenyi and, including the effects of discontinuities and augmented beams, were straightforward; for the worst loaded beam, the penultimate

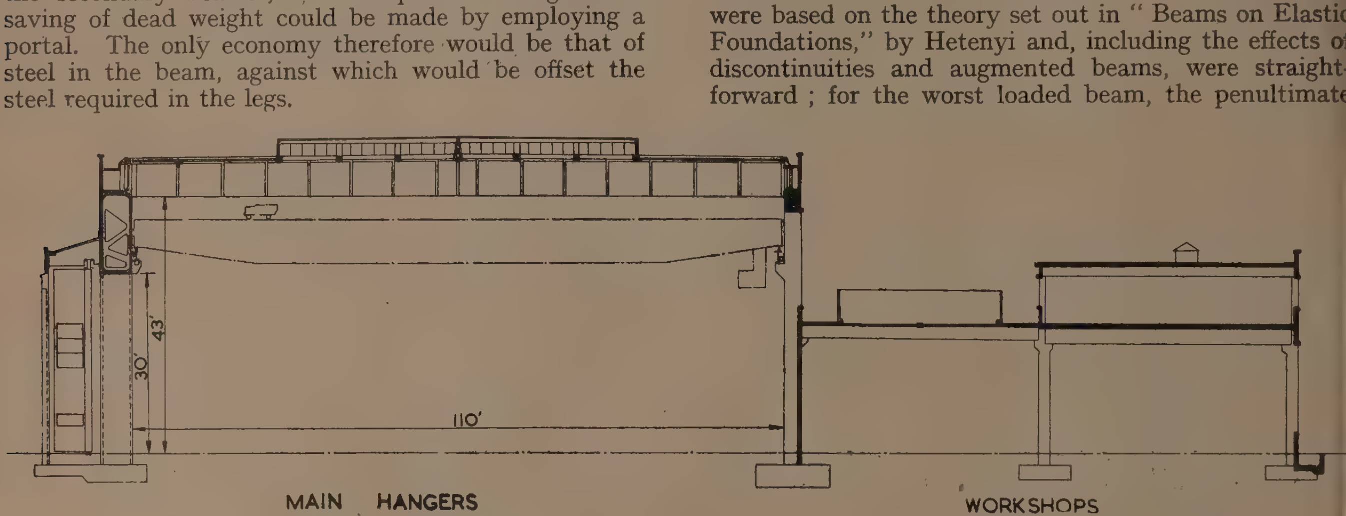


Fig. 1. Section of half-structure showing general arrangements

The door beam was thus to be box-shaped and simply supported. Preliminary calculations showed that 4 in. webs would carry quite comfortably all the shear stresses caused by bending and torsion and would be sufficiently stable against buckling if diaphragms were provided at each secondary beam bearing, i.e., at 15 ft. centres.

The next question was, where to place the prestressing cables. Precedent exists for placing the cables in the central void (hangars at Melsbroeck, Brussels), and the idea has its attraction, particularly since with the type of cables employed (Freyssinet cable) the protection problem can be neatly solved. This arrangement of cables, however, has serious disadvantages, notably:

(1) If the cables are not solid with the webs, the prestressing force is *external* and tends to produce buckling; the webs must thus be made thicker, adding very considerably to the weight of the beam and to the quantities of steel.

(2) If the cables are not in the line of the webs, the end blocks are subjected to very heavy shear forces and must in consequence be substantial and heavily reinforced. With the cables lying within the webs, however, it is sufficient at the beam ends to thicken up the webs slightly and provide sufficient bracing to counter local buckling due to the support reaction.

(3) Unless the cables are led into and out of the bottom flange, there is an appreciable loss of eccentricity which reacts not only on the steel quantities but on the concrete section as well.

The main beam was therefore to be simply-supported and box-shaped with the cables embedded in the concrete walls. Exact calculation started from this point and the results will be found below.

Secondary Beams

The largest part of the load on the secondary beams being that due to the cranes, means were sought of distributing it, and a triangulated prestressed beam

beam, the point load was reduced from 22.5T to 9.4T, to which must be added the weight of the transverse beam which came to 2.3T. The point load had thus been halved.

The secondary beams thus being united by a very stiff transverse beam, conditions of transverse and torsional stability were greatly improved, and a simple I section, with occasional stiffening diaphragms was found satisfactory.

The scheme having been drawn up on these lines, the authorities decided to replace the two cranes by one large one spanning clear from the door to the wall. This removed all point load from the secondary beams and changed their character entirely; the reasons which had seemed to demand a solid web no longer existed. It was too late, however, to recast completely the scheme for the secondary beams and in consequence they were modified for the lighter loading by removing the bottom flange and reducing the number of cables. The T beam thus resulting is of a form not often economic in prestressed concrete, unless dead load is preponderant, which is not the case here; an examination of the stress values below, however, will show that the strength of the concrete is fairly fully mobilised. The simplicity of shape aided casting.

Lateral stability required attention; the breadth/span ratio of the beams was 1/36. While this would be sufficient during short periods unloaded, it would not be sufficient under full load and over long periods. Arrangements were made therefore to provide lateral bracing by means of the beams carrying the lanterns. Similarly the web and flange were very slender and it was necessary to unite them by fairly frequent diaphragms.

(IV) Construction Procedure and its Bearing on the Design

The scheme being prepared in close collaboration with the contractors, Messrs. Holland & Hannen and Cubitts

Ltd., erection problems were dealt with, as they should be, prior to detailed design to the benefit of the works as a whole. The construction aspects which had a bearing on design were as follows :

(1) Main Beams

The first problem was—to precast or not to precast. The contractors decided against on the following grounds :

(a) The sectional area of the main beam is 2,035 sq. in. The segments could not reasonably weigh less than 5T and this, in view of the slenderness of the sections, meant delicate handling.

(b) Lacking experience of such large segments, it was difficult to foresee what degree of exactitude could be relied upon in overall dimensions, positioning of holes, etc.

Accordingly it was thought prudent to cast the walls in situ.

The diaphragms, however, presented a different problem. One was required every 15 ft. to carry the concentrated load from the secondary beam ; it was a simple matter to combine with this diaphragm a cantilever to carry the canopy and a bracket to carry the crane beam, thereby concentrating in this element all the special features met with along the length of the beam. More, by precasting these diaphragms with holes to carry the main beam cables, they would greatly simplify the work of fixing the steel and the formwork. So it was done ; special diaphragms occurred at the beam ends to carry the anchorage cones and all these precast elements were placed on the falsework, the cables threaded through them (the cables required little other support) and the formwork fixed. The in situ concreting was thus reduced to the flanges and the webs.

(2) Secondary Beams

These beams could have been constructed in any of three ways :

(a) *Cast in situ.* This was obviously to be avoided if possible. The scaffolding and shuttering alone, not to speak of the labour for erecting, dismantling and displacing, would have been a very costly item.

(b) *Cast in one piece elsewhere* and handled into the final position. This was practical ; the handling operations were in no way too onerous since the beams would not weigh much more than 25T. The beams could either have been cast on the ground near their final position and lifted up or cast on a staging and rolled into place along the top of the main beams. Nevertheless, to maintain the necessary progress, several sets of formwork would have been needed.

(c) *Cast in segments.* This was the method finally decided upon. Its advantages were the following :

(i) Greater precision in casting would be obtained and better control of quality. Inspection of each unit would reveal any defects ; should such appear, to reject a unit was a matter of much less gravity than to cut out a section of a beam cast in one piece.

(ii) From the progress point of view, it was convenient to be able to continue the casting of the secondary beams away from the main area of the works and stack the segments ready for use when required.

(iii) From the point of view of direct economy, the saving of formwork was most attractive and in fact segmental construction fully justified itself from this point of view. It is of interest to note that a total number of 122 beams, containing 3,074 elements, were cast in nine moulds.

(iv) The difficulties of casting concrete down a 4 in. web around a line of cables running horizontally, difficulties which were accepted on the main beams, could be avoided by casting the segments on end, when the concrete would be placed along, rather than across, the line of the cables.

The main bearing upon design of the decision to employ segmental construction related to the stiffeners. It was obvious that so slender a web and flange would need frequent diaphragms ; such diaphragms would greatly complicate formwork were the beam to be cast in one piece. With segmental construction, however, to cast a diaphragm at one end of each segment so far from presenting any difficulties had certain positive advantages ; it stiffened each element during handling and provided an open mouth into which to drop the concrete

Plate 3. First main beam under test

during casting as well as stabilising the beam very effectively.

(V) Practical Details

In both beams, Freyssinet anchorages and cables were used of the 12 × 0.2 in. type. The cables were sheathed in light steel tubes whose exact nature varied according to the supply position. After stressing and anchoring, the cables were pressure grouted with 1 : $1\frac{3}{4}$ cement-sand mortar and the cone recesses sealed off with mortar.

Main Beams

The main problem here was the disposition of the cables. The solution adopted was to group them in the centre as shown in sketch with one layer of 21 spaced evenly right across the bottom flange with a cover of 1 in., and two layers of 6 and 4 above them, with $\frac{1}{2}$ in. clear spacing, in each corner. This gave a distance of the centroid of the steel of 2.75 in. above the soffit. As the cables left the centre section, the outside vertical group of cables was swept up into the web, and the next vertical group was bent outwards to take its place and swept upwards in its turn. The phase-length, as it were, of this succession of cables was 15 ft. ; a diaphragm thus marked the point at which cables ceased to turn outwards and commenced to bend up. There was thus a double movement of cables outwards from the bottom flange and upwards into the web ; in the support section the process reached its conclusion with seven cables in the bottom flange and 17 in each web. This gave a height of centroid above the soffit of 67 in., and a very well distributed anchorage pattern. The locus of the cable centroid was approximately parabolic. While the double curvature would seem at first sight to lead to high friction losses, in fact it was not so since the cables with large vertical curvature had only a minute horizontal curvature and the cables with maximum horizontal displacement in fact only bent out 15 in. in 75 ft. with no vertical displacement at all. The bursting force caused by this slight outward spread was more than compensated by the reinforcement of the diaphragms.

At one end the beam rested on a very rigid box column. The support was composed of a steel plate bearing and the beam was held down by two Freyssinet cables anchored by hooked anchorages in the shaft of the column. This bearing was directly under a standard diaphragm and the beam projected 2 ft. 6 in. beyond this diaphragm and terminated in a special diaphragm, carrying the anchorage cones. Over this length the beam webs were thickened up to 12 in.

It will be seen from the drawings that there was a space of 30 ft. between the support axes of adjoining main beams ; in this 30 ft. had to be fitted the expansion joint and room had to be left for the stressing jacks to operate since it was thought undesirable to stress a beam of this nature from one end only. In consequence, whilst at one end the beam projected only 2 ft. 6 in. beyond the support axis, at the other it projected 15 ft. in cantilever. The cable inclinations were continued across this 15 ft., which resulted in some of the cones lying in the top surface of the beam. This produced no difficulties. This end of the beam rested on a flexible column, the bearing being similar to that at the other end.

The intervening 12 ft. 6 in. was cast in situ in reinforced concrete, the prestressing wires protruding from the fixed end being employed to bond in the reinforced work.

The expansion joint thus ran up the back of the flexible column, along the soffit of the cantilever and up the end of the cantilever.

Thirty-three cables were stressed up when the concrete had set sufficiently ; the remaining eight cables were stressed as soon as the secondary beams in the centre of the span had been placed.

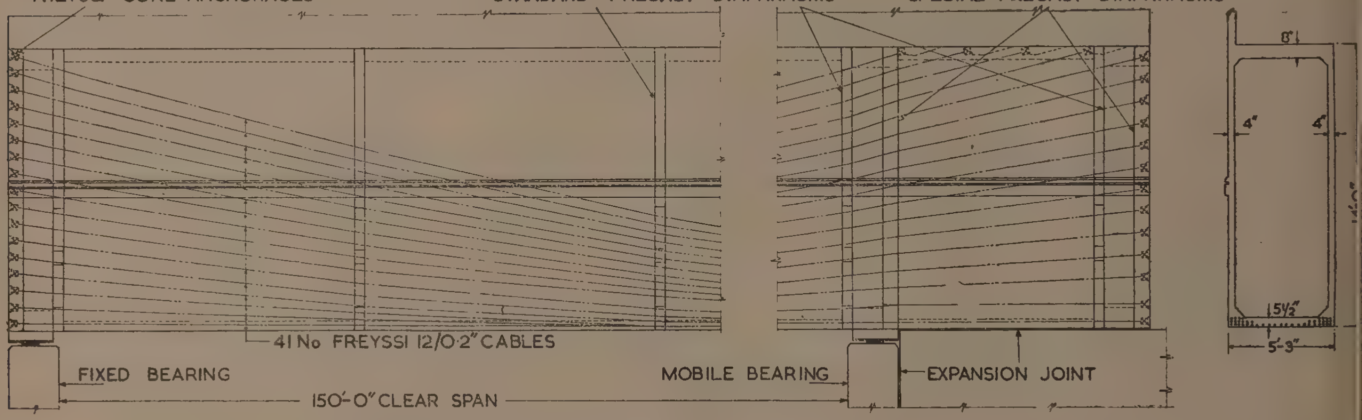


Fig. 2. Elevation and section of main beam

Secondary Beams

The cable trace in the secondary beams was complicated by the necessity of providing a number of holes in the beam web to carry lighting and other services. The employment of eight cables at a distance of 14 in. from the beam soffit to their centroid gave considerable flexibility in choosing the trace of each cable and as a result it was possible at the same time to preserve a reasonable shape for each, to avoid the holes and obtained a roughly parabolic shape of the centroid locus.

The joints were $\frac{3}{4}$ in. wide and were filled with a 1 to 1 sand/cement mortar mixed earth dry and packed in hard with a hammer and chisel. The cables were protected from this mortar by a short length of tube passing across the joint and extending into bell-mouths cast in the ends of the adjacent elements. The cables were stressed up 24 hours after the packing of the joints. One incident has to be remarked upon in this connection. One of the secondary beams having been successfully stressed, one of the joints failed about 12 hours after when the cable ducts were being washed through with water prior to being injected. The causes of this remain obscure, though it was probably due to the effect of the flow and pressure of water on a joint whose set for some reason was delayed. Extensive examina-

ion and testing by the contractors of the concrete in he joint and in the elements failed to produce any nore likely explanation. At any rate, the stress was eleased, the damaged units replaced and the beam tressed up again without incident.

The end element carrying the anchorage cones was a airly bulky piece of lightly reinforced concrete. It was otched on its top surface to carry the gutter and on ts bottom surface to permit the flange to project below he upper level of the main beam. The beam end was equired to be broad to prevent the overturning of the eam and the top flange had to be carried through to the gutter to support the deck ; a simple though heavy shape

Plate 4. Assembly of secondary beams

was chosen to satisfy these needs and to add robustness to the extremities of a beam which would undergo a fair amount of handling. The bearing surface was recessed and fitted over a reinforced concrete ridge which thus gave the beam a mechanical key with the supporting structure at either end ; as has been remarked above, the columns at the back of the building were sufficiently flexible to permit of temperature movements without overstraining.

Roof Decking

Pretensioned purlins were bolted down on to the secondary beams through holes cast in the flanges. A precast kerb was then bolted along the beam flange between the purlin ends and lightweight aluminium troughing laid on top, the whole being finished off with fibre-board and a bituminous felt finish, the felt being brought over the kerb and on to the beam flange in such a way that the beam flange formed a gutter.

The lanterns were formed by triangular shaped precast beams placed on the secondary beams. The centre lantern beam coincided with a special diaphragm ; bars projected from each and in situ concrete cast around them ensured a joint of adequate strength.

(VI) Design

The loading specified was in general as in C.P.3 Chap. V (1944). For wind loads, p was to be taken as 25 lb./sq. ft.

The clauses in the specification covering the design of the prestressed concrete work were in close agreement with current practice ; the stresses specified were as follows :

(a) Concrete.

Compressive bending stress : 2,000 lb./sq. in.

Tensile bending stress during erection in beams cast in one piece : 200 lb./sq. in.

Tensile bending stress in any beam under any working condition : Nil.

Principal tensile stress under shear : 150 lb./sq. in.

These stresses would require a concrete of not less than 6,000 lb./sq. in. crushing strength at 28 days.

(b) Steel.

Final working stress in tensioned steel : 60 per cent. of the U.T.S. after allowance for relaxation.

The allowances for relaxation were specified as follows :

(a) Concrete ; creep strain = $(0.3 \times 10\text{-}6 \times$ average prestress) per unit length.

(b) Concrete ; shrinkage strain $= 200 \times 10^{-6}$ per unit length.

(c) Steel ; creep loss = 5 per cent.

(d) Anchorage strain = 1.5 per cent. of steel stress $\div$ length in feet.

These allowances were later modified slightly as follows :

It was agreed to replace the expression for anchorage strain by a flat allowance of 3/16 in. for each Freyssinet cone, a figure which is regularly found in practice, and in view of the fact that none of the concrete was stressed up prior to 28 days of age, the shrinkage strain was neglected. These various provisions gave a total loss of the order of 15 per cent., which is again in line with current practice. The U.T.S. of the steel varied between 105T and 110T per sq. in. 108T/sq. in. was taken as an average, which gave a final force of 55,000 lb. max. per 12×0.2 in. dia. wire cable.

Main Beam

(1) Vertical Bending.

Bending moments are :

Own weight	84,400 kips. in.
Dead load of roof, canopy, crane beam and secondary beam	133,600 kips. in.
Live load (a) positive (snow and crane load)	73,600 kips. in.
(b) negative (wind suction) ...	58,200 kips, in.

In the preliminary project, the cable holes in the beams were neglected—the influence of such neglect is small and favourable to security. The contract having been awarded, however, a more exact analysis was made and the section minus holes employed for the prestress, own weight and dead load calculations, and this section augmented by the steel area multiplied by a modular ratio of 8 was employed for the live loads. These results are given below. As will be seen, there is an appreciable margin of compression left under all extreme cases ; the dimensions of the section and the number of the cables were left unaltered, however.

Section prior to grouting is (all figures are in inch units) :

$A = 2035$; $I = 7{,}528{,}000$; $v_1 = 76.5$ in. ; $v_2 = 91.5$ in. ; $z_1 = 98{,}400$; $z_2 = 82{,}300$.

Eccentricity of cable centroid = 88.75 in.

Prestresses are :

$$\frac{55{,}000 \times 41}{2035} + \frac{55{,}000 \times 41 \times 8645}{82{,}300} = +3540$$

i.e., 2760 at 1st prestress of 32 cables and 780 at 2nd. of 9 cables.

$$\frac{55{,}000 \times 41}{2035} - \frac{55{,}000 \times 41 \times 86.45}{98{,}400} = -920$$

i.e., —718 at 1st prestress of 32 cables and —200 at 2nd of 9 cables.

Load stresses are :

Own weight : + 858 and —1025
Dead load : +1360 and —1620

Section after bonding of steel is :
equivalent $A = 2143$; $I = 8{,}338{,}450$; $v_1 = 87.1$; $v_2 = 86.9$ in.
$z_1 = 102{,}600$; $z_2 = 96{,}000$.

Load stresses are :

Live load, positive : +718 and —768
Live load, negative : —567 and +606.

The sequence of stresses may be represented in tabular form as follows :

Min. vertical shear is

Own weight	=	187,500
Dead load	=	297,000
Wind load	=	129,000
		365,000

Vertical upward component of prestress force = 345,000.

.... Max. Resultant Shear 276,000.

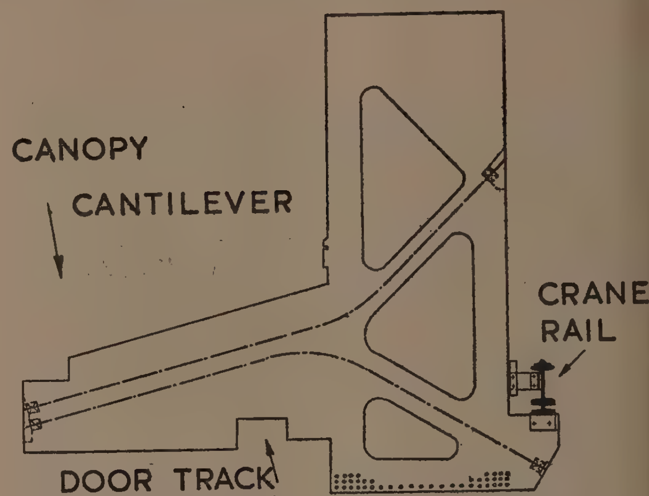


Fig. 3. Elevation of pre-cast diaphragm in main beam

D.W.	1st P.S.	Σ	D.L.	Σ	2nd P.S.	Σ	+L.L	Σ	—L.L.	Σ
+858	—718	**+140**	+1360	**+1500**	—200	**+1300**	+718	**+2028**	—567	**+733**
—1025	+2760	**+1735**	—1620	**+115**	+780	**+895**	—768	**+127**	+606	**+1501**

(2) Horizontal Bending

The beam was assumed to carry horizontal bending caused by the full wind pressure acting on the whole of one external vertical surface.

The moment thus produced was 14,980 kips. in.

The modulus of the section about a vertical axis being 43,000, this gave a stress of ±350 lb./sq. in.

This load acts in conjunction with the wind suction on the roof ; the worst case thus occurs when the doors are closed and the roof is completely unloaded, when the worst stresses are :

		H. wind	Σ
Upper surface ...	+733	±350	+1083 and +383
Lower surface ...	+1501	±350	+1851 and +1151

(3) Shear

(a) Due to vertical loads :

Max. vertical shear is

Own weight	=	187,500
Dead load	=	297,000
Live load (crane)	=	33,600
(snow)	=	103,500
		621,600

Weakest point is at top of web where prestress is smallest ; shear stress here = 165 lb./in.^{2}

(b) Due to horizontal loads :

Max. horizontal shear = 33 kips.
Shear stress at top of web = 61 lb./per sq. in.

(c) Due to torsion :

The heaviest torque is caused by the wind blowing from the back wall towards the open door when the roof is unloaded and the canopy is loaded to its maximum.

This torque is composed of the following actions :

Suction on parapet	=	59,000 lb. in.
Pressure on back wall transmitted by secondary beam to upper flange of main beam	=	814,000 lb. in.
Load on cantilever	=	9,384,000 lb. in.
Total	=	10.26×10^6 lb. in.

Batho-Bredt formula gives $q = \dfrac{T}{2At}$

where T = torque. A = Area enclosed by median line. t = wall thickness.

This gives $q = 135$ lb./sq. in.

(4) Resultant Principal Stresses

The effects of vertical and horizontal bending and of torsion produce shear stresses which on one surface can all act in the same sense ; the maximum conditions for each are not all compatible, however. The worst combination is the following :—

Vertical shear under full load and wind uplift = 85 lb./sq. in.
Max. horizontal shear = 61 lb./sq. in.
Max. torsional shear = 135 lb./sq. in.

281 lb./sq. in.

The compression stress at the root of the top flange fillet is 780 lb./sq. in.

The principal tensile stress is thus

$$= \tfrac{1}{2}(\sqrt{780^2+4.281^2} - 780)$$
$$= 90 \text{ lb./sq. in.}$$

In view of the various incalculable effects of shrinkage, temperature variation, etc., which lead to shear stresses

Plate 5. Typical pre-cast diaphragm

at the corners of hollow box beams, it was thought wise nevertheless to reinforce these corners with M.S. bars.

Secondary Beams

(1) Bending

(a) Vertical.

Section prior to grouting is :

$$A = 406 \; ; \; I = 216{,}420 \; ; \; v_1 = 24.8 \; ; \; v_2 = 47.2 \; ;$$
$$z_1 = 8{,}780 \; ; \; z_2 = 4{,}580.$$

Eccentricity of cable centroid = 33.2

Bending moments are :

Own weight = 9,000 kips. in. giving stresses of +1025 and —1875.

Prestresses are :

$$\frac{53{,}000\times 8}{406} + \frac{53{,}000\times 8\times 33.2}{4580} = 4120$$

$$\frac{53{,}000\times 8}{406} - \frac{53{,}000\times 8\times 33.2}{8780} = -570$$

Section after grouting, with bonded steel, is : equivalent $A = 427.2$; $I = 240{,}000$; $v_1 = 26.2$;

$$v_2 = 45.8 \; ; \; z_1 = 9160 \; ; \; z_2 = 5230.$$

Bending moments are :

Dead load = 4,950 kip. in. giving stresses of +540 and — 948

Live load (positive) = 5,320 kip. in. giving stresses of +580 and —1010

Live load (negative) = 7,130 kip. in. giving stresses of —778 and +1360.

Whence stress sequence is :

O.W.	P.S.	Σ	D.L.	Σ	+veL.L.	Σ	—veL.L.	Σ
+1025	—570	**+455**	+540	**+995**	+580	**+1575**	—778	**+217**
—1875	+4120	**+2245**	—948	**+1297**	—1010	**+287**	+1360	**+2657**

It is permissible to increase stresses due to wind loads by 33⅓ per cent. ; the stress under maximum uplift is thus not excessive.

(b) Horizontal

An investigation of the horiztonal bending effect due to wind drag and the wind pressure on the lanterns and various projections in the roof was made in accordance with C.P.3. Chap. V (pp. 17 and 18).

Assuming no fixity in the horizontal plane at the supports, this gave a moment of 287 kip. in., which produces a stress of ± 334 lb./sq. in.

The conditions producing drag and horizontal pressure cannot coincide with overall suction. The worst cases for stress in the top flange are as follows :

Dead load alone : $+995 \pm 334 = +1329$ and $+661$

Dead load and live load $+1575 \pm 334 = +1909$ and $+1241$.

(2) Shear

Max. shear force is :

Own weight	=	26,500
Dead load	=	11,750
Live load (+*ve*)	=	16,800
		55,050

Min. shear force is :

Own weight	=	26,500
Dead load	=	11,750
Live load (—*ve*)	=	21,000
		17,250

Vertical component of prestress at end section = 28,000

$\therefore$ Max. Resultant shear force = 27,050

Shear stress at root of web = 102 lb./sq. in.^{2}

Horizontal compression at root of web = 640 lb./sq. in.

$\therefore$ Resultant principal tensile stress

$$= \tfrac{1}{2}(\sqrt{640^2+4.\ 102^2}-640)$$
$$= 16 \text{ lb./sq. in.}$$

Stresses due to horizontal shear were negligible.

(3) Torsion

A check was made on the torsional strength of the secondary beams when the bay on one side of the beam was fully loaded and that on the other unloaded. Without taking any account of the stiffness of the lantern frame, it was found that the principal tensile stress would be 126 lb./sq. in. It was thought that further investigation was unnecessary.

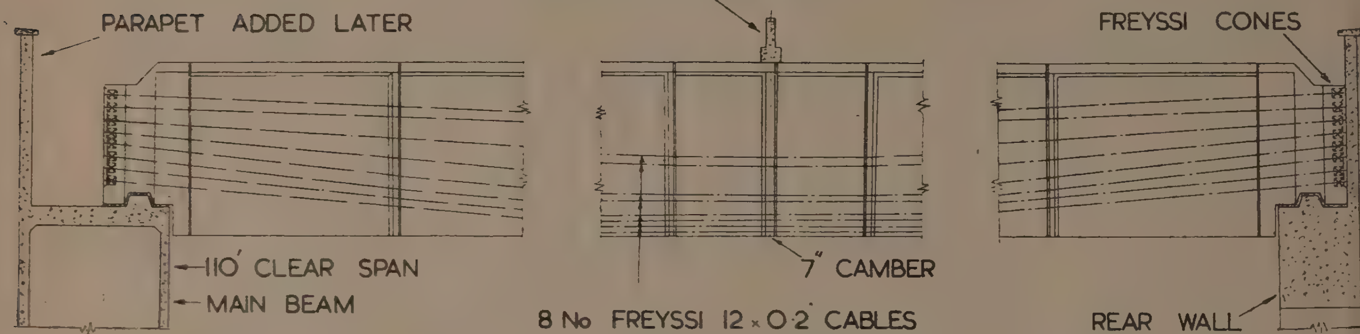


Fig. 4. Elevation of secondary beams

Plate 6. Main beam under construction

Quantities

It is of interest to note that the total quantities of material in the main and secondary beams when divided by the area of roof supported give the following figures

(a) H.T. Steel ... 1.05 lb. per sq. ft.
(b) Mild Steel ... 0.98 lb. per sq. ft.
(c) Concrete ... 3.75 in. average thickness.

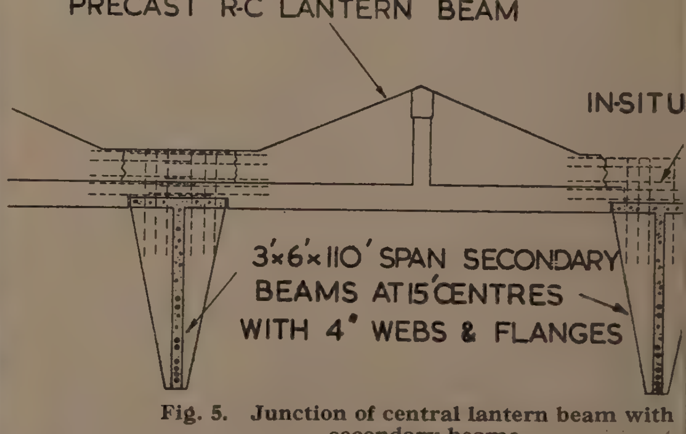


Fig. 5. Junction of central lantern beam with secondary beams

(VII) Tests

Two secondary beams and one main beam were tested with the full design load. The results were as follows

(a) Secondary beams.

The beams were placed together on supports on the ground and braced together in such a manner as to simulate the conditions of service.

The deflections of one beam were measured by deflectometer, those of the other by a scale. The design load was 24T per beam ; the deflection under this load of the former was 1.510 in., of the latter $1\frac{5}{8}$ in. Considering the beam measured by the deflectometer, the average settlement of the supports was 1/16 in. ; this gives a deflection under load, of 1.447 in., which corresponds to a modulus of elasticity of the concrete of 5,180,000 lb./sq. in. The residual deflection after unloading was $\frac{1}{8}$ in.

Plate 7. Roof structure prior to cladding

(b) Main beam

The main beam was cast with a camber of 5 in. ; after 33 cables had been stressed this camber increased to $6\frac{1}{8}$ in. The secondary beams were then added and the remaining eight cables stressed, when the camber fell to 6 in. The beam was then loaded with 288T of pig-iron to bring the load up to the maximum design figure. The application of this load was completed at 3.15 p.m. on a Friday afternoon and the load was left on over the week-end. The deflections recorded were as follows :

Friday	3.15 p.m.	1.1335 in.
Saturday	10.15 a.m.	1.150 in.
Sunday	10.15 a.m.	1.1795 in.
Monday	10.15 a.m.	1.249 in.

The load was then removed and the residual deflection was 0.214 in., a figure which fell to 0.1955 in. after a lapse of 1 hr.

The deflection of 1.249 in. corresponds to a modulus of elasticity of the concrete of 5,210,000 lb./in.

Book Review

Building and Civil Engineering Plant, by Spence Geddes. (London : Crosby Lockwood.), 1951. 10 in. × $7\frac{1}{2}$ in. 297 pp. 30s.

The first thing which strikes a reviewer of this book is the logical and orderly arrangement of the sections into which the author has divided each subject.

Reading through it from cover to cover, as only a reviewer would do, the repetition of the same lines of approach to each type of plant gives a feeling of almost monotonous regularity. Yet this very regularity commends itself to the seeker for information because the same kind of details are given in the same kind of way in every section and the proper place to look for any particular information becomes self-evident.

It is, therefore, a first-class reference book which covers its very large subject more than adequately.

The scope of the book includes references to practically every kind of building and civil engineering plant which a contractor is likely to use, describing plant used in connection with agricultural works, concrete making and placing, muckshifting, pile driving, pumping, trenching and transporting, to refer only to a few of the sections.

Preceded by four sections on considerations involved in purchasing plant, its efficient operation and its cost, the remainder of the book is divided into nineteen further sections, each of which deals with a particular kind of plant.

The descriptions of all types of plant are very comprehensive, giving details of construction, including details of power units, types of fuel required and rates of consumption, followed by details of the application, operation and correct types of plant to be used to fulfil the various requirements of a contract.

Typical examples are given showing how to estimate the size of plant required to do a given job in a given time and these are followed by tables giving information on output, apportionment of labour and ancillary equipment necessary for the plant to be worked to its maximum efficiency.

The book is profusely illustrated with photographs, reproduced drawings and tables and contains also many useful comments upon maintenance and repair. Cross referencing between related sections is very thorough.

This book has clearly been compiled by one whose practical knowledge of the subject is wide, and it should prove a most useful source of information for both consultant and contractor.

D. F. W.

Notes of Interest

Members of all grades are invited to submit " practice notes " of individual operations such as those described below, whose size and scope are not perhaps sufficient to justify a full-length article or paper, but which possess features of special or general interest to the Institution.

Strengthening of Old Reinforced Concrete, Timber and Steel Beams on a Recent Contract

By John Faber, B.Sc., A.M.I.C.E., A.M.I.Struct.E.

Introduction

These notes describe one particular part of the work recently completed at Messrs. Spillers' Soya Mills at Atlantic Wharf, Cardiff (Fig. 1), where old warehouse and silo buildings have been modified and re-equipped.

Warehouse " A," containing five floors, was built about 1880 with external load-bearing walls of stone, timber floor beams and cast-iron columns. Warehouse " B," with eight floors, was built later, with external load-bearing brick walls, steel floor beams and cast-iron columns. The silos were built in 1913 of reinforced concrete, and include eight floors in the Receiving House.

The buildings were originally used principally for warehousing, and then for many years stood empty with windows broken and the roof, leaking, exposed to severe weather conditions. Now the buildings are equipped for the extraction of oil from soya beans and the manufacture of numerous by-products.

Scope

All the existing buildings were found to be unsuitable in varying degrees for the new duties required of them, particularly as regards the carrying capacity and condition of the floor beams. The notes describe how these beams have been strengthened.

No claim is made here for originality, but it is hoped that it may be of interest to have a record in the Institution Journal of this type of work since it undoubtedly forms a part of present-day structural engineering, whether as a result of war damage or the need to economise in new construction due to current restrictions.

Concrete Beams

Fig. 2 (a) is a typical floor plan of the reinforced concrete Silo Receiving House showing the old arrangement of beams and the numerous openings left through the 3 in. concrete floor slab to suit chutes, elevators, etc. Many of these openings had clearly been cut at different stages in the history of the building, and there was evidence too of earlier holes which had been filled in. It was not clear what reinforcements could be relied upon as remaining.

The secondary beams were only 7 in. × 4 in. nett under the 3 in. slab, and did not occur where the new loads had to be supported. Furthermore, many secondary beams passed across positions required for new openings.

The main beams were more fortunately placed, but in many cases longitudinal cracking and spalling of the soffits showed that rusting of the reinforcements was taking place so that bond was precarious and progressive splitting likely. In other cases the main beams were not suitably reinforced to carry the new loading arrangements.

Trial areas of the slabs were broken out. Some of the beam stirrups were found to have rusted right through at slab soffit level, and there was no key between the slabs and beam ribs.

The external walls of the Receiving House were only 4 in. thick for the full 102 ft. height of the building, stiffened only by pilasters and with no beams at the floor levels. The walls were generously provided with window openings. For these reasons it was considered unwise to loose the tying effects of the main beams which framed into the stiffer construction of the silo bins.

The strengthening was carried out as follows: The floors were treated one at a time. First, the floor slab was demolished: this came away from the beam ribs

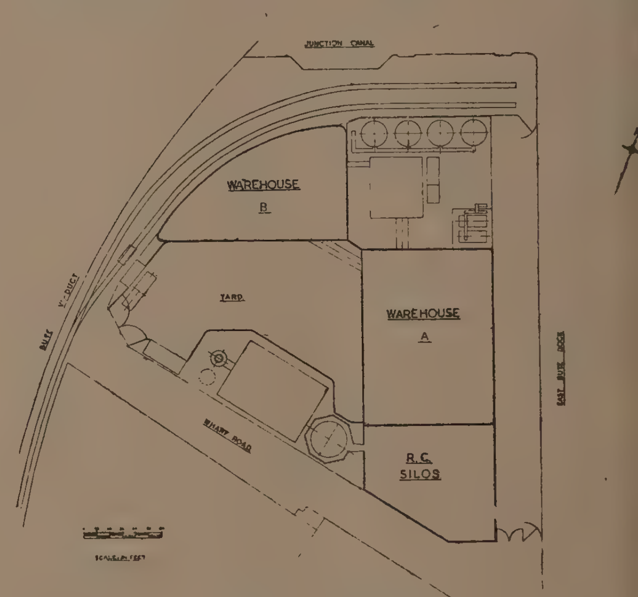


Fig. 1. Site plan

easily. The slab reinforcements which bonded into the surrounding walls of the Receiving House were cut off two feet from the walls to assist later in tying the new work to the old.

The secondary beams were in most cases also demolished, though it was possible to retain a few of these to carry local loads.

Great care was required in dealing with the main beam ribs, since the bottom reinforcements were to be the life-line in tying the 4 in. outer walls.

Where the main beam ribs showed no signs of longitudinal cracking, they were retained, though the top $1\frac{1}{2}$ in. was carefully chipped away to provide bearing for a new $4\frac{1}{2}$ in. slab. In many cases, however, the uncracked ribs required strengthening to carry additional loads and this was done by providing additional reinforcements as shown typically in Fig. 3 (a). The widths of the strengthened ribs were 4 in. greater than the original

ribs, and bearing ledges 2 in. × 2 in. were cut in the existing pilasters so that the additional width of rib could transmit a useful share of vertical shear. The concreting of these "strengthened" beams was carried out by pouring stiff grout in at one side of the beam shutters and vibrating the shutters until the grout worked its way under the old beam rib and started to rise up the other side. The remainder of the rib was concreted using $\frac{3}{8}$ in.—$\frac{1}{4}$ in. aggregate. The mix was 3 : $1\frac{1}{2}$: 112 lb. bag.

Where the main beam ribs were cracked and spalling, the soffits were firmly propped at the ends and the rib then cut away carefully with pneumatic hand drills.

A new $4\frac{1}{2}$ in. slab was cast over the main beam ribs, Fig. 2 (b), being properly keyed and tied to the beam ribs by the new stirrups, and to the old walls by the original slab continuity bars which had been preserved. The $4\frac{1}{2}$ in. slab was reinforced as required to carry the new machine loads and provided with holes and pockets as necessary.

At the two ends of the Receiving House, where a bearing was required for the new slab on the bin walls, a continuous 4 in. × 3 in. × $\frac{1}{2}$ in .steel angle was provided bolted to the walls with special wedge-tailed anchor bolts bearing between two tapered steel plates (Fig. 3 (c). This saved the need to drill right through the bin walls

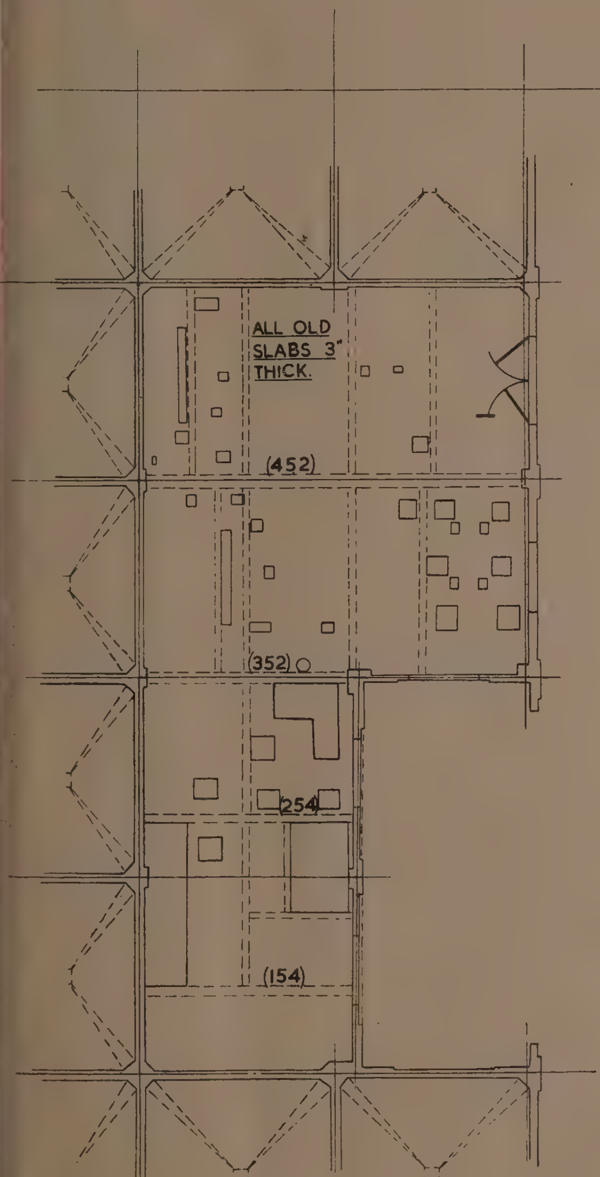


Fig. 2 (a). Silo : Old floor

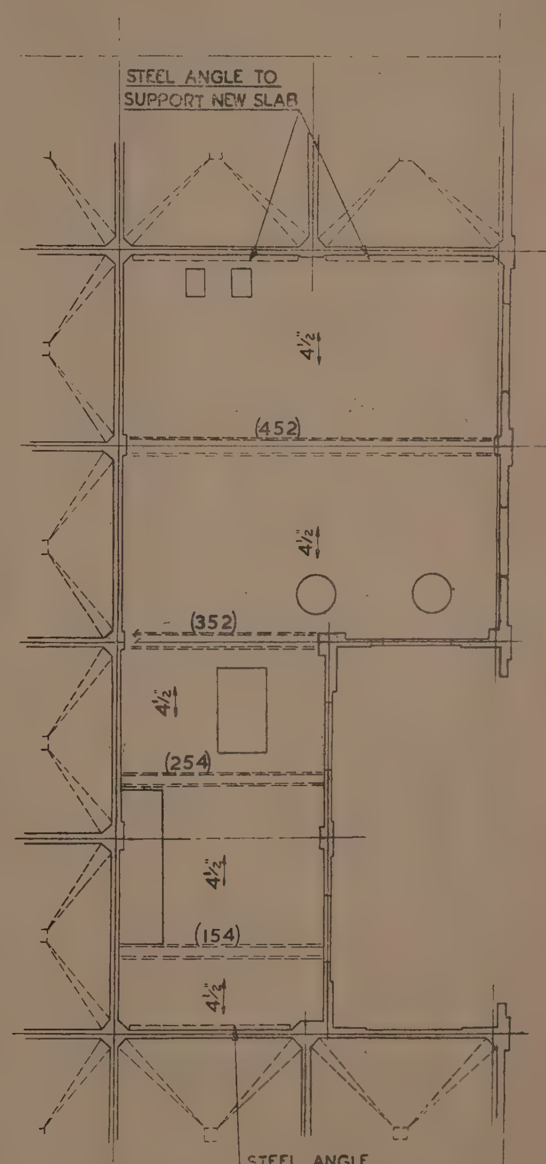


Fig. 2 (b) Silo : New floor

A triangular wedge as shown in Fig. 3 (b) was preserved at each end to act as a bracket and transmit vertical shear in the completed beam, and the sides of these wedges were bush-hammered. 2 in. × 2 in. bearing ledges were also cut in the pilasters as described in the previous paragraph. The existing longitudinal beam reinforcements were preserved to act as ties as before described. but additional reinforcements were provided to take the bending stresses and lift the end shears on to the triangular wedges. The beams were then "reconstructed" as in normal construction.

which would have entailed scaffolding from the bin hopper bottoms to make good the areas spalled by the drilling and to enable the bolts to be threaded through the walls.

Timber Beams

The timber main beams in Warehouse "A" are generally about 12 in. × 14 in. cross-section on about 13 ft. span. Where these would be overstressed by the loads of new machines on the floors their strength has been augmented by pairs of steel channels (one on either side) carried by steel "saddle" joists sitting astride the

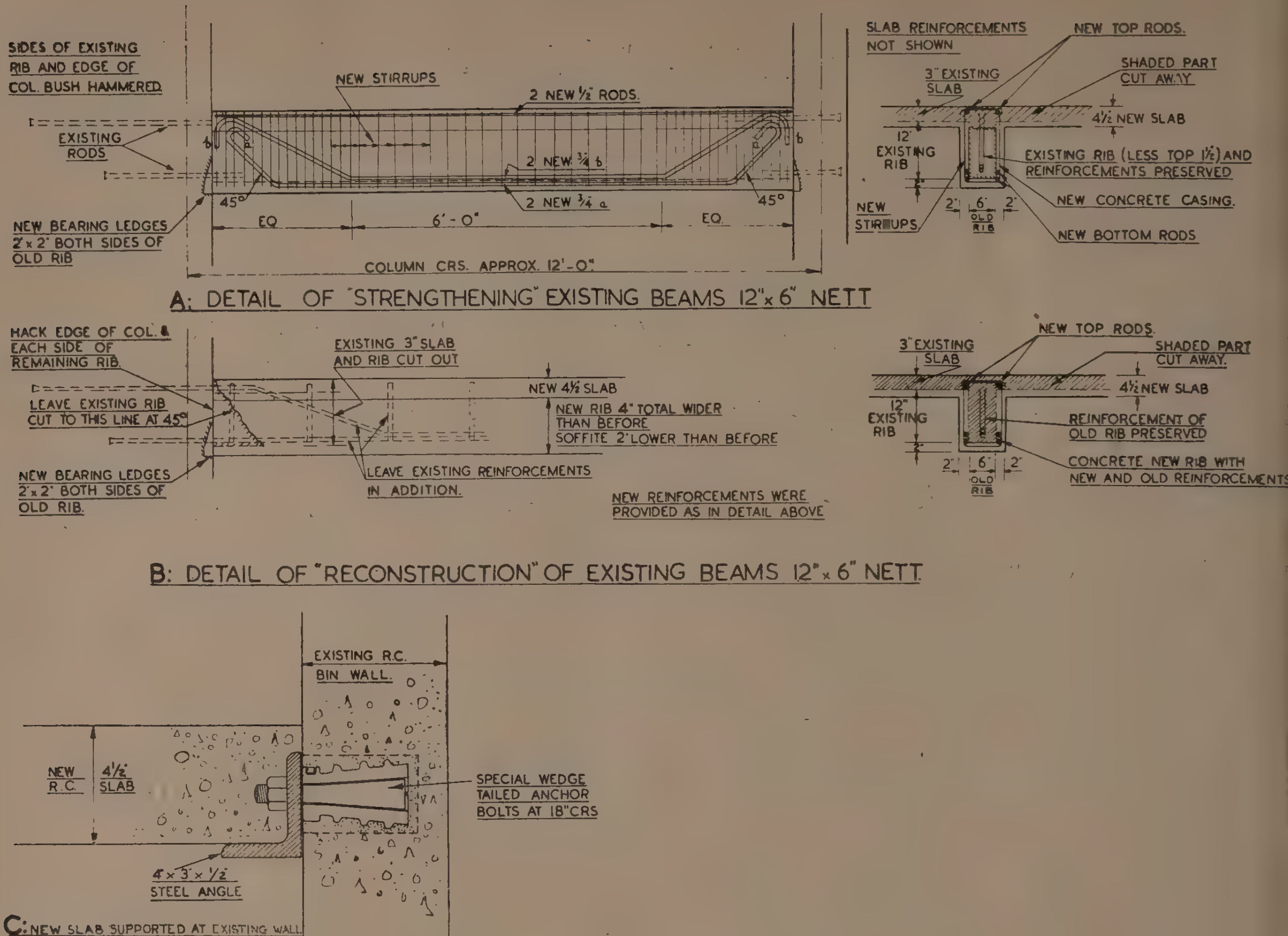


Fig. 3. Repairs to R.C. floors in silos

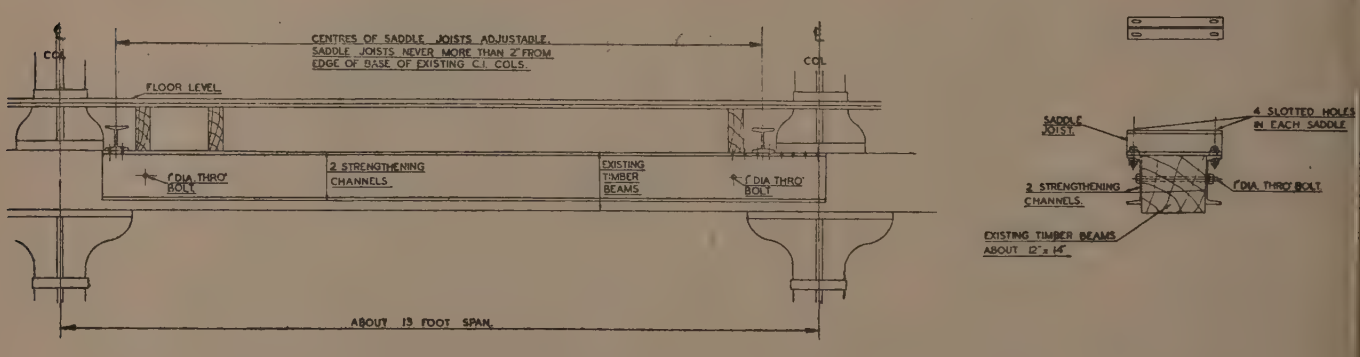


Fig. 4 (a). Strengthening of timber beam

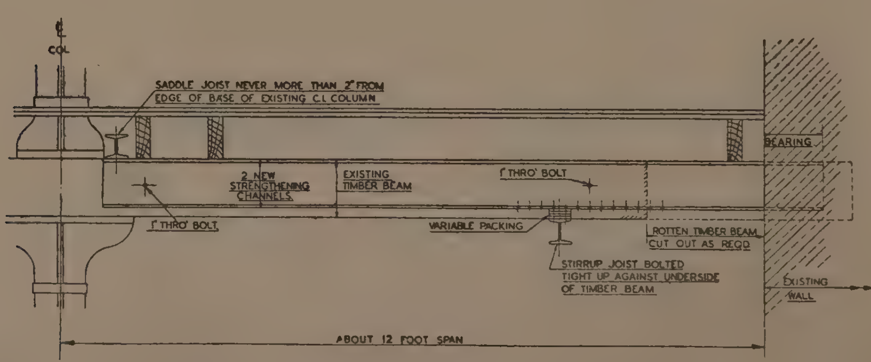
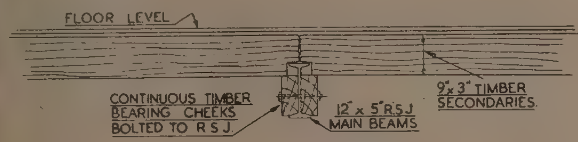


Fig. 4 (b). Support of rotted timber beam

timber beams at the supporting cast-iron columns. The cast-iron columns are of cruciform section, and were found to have an adequate margin of strength. The brackets at the heads of the columns always project further than the brackets at the feet of the columns over, so that the saddle joists produce very little bending in the timber beams. See Fig. 4 (a).

By a suitable choice for the stiffness of the strengthening channels it has been possible to make the steel and timber share the loads, thus making for economy.

Owing to the varying spans of the main beams (there seemed to be no two cases quite alike), the strengthening channels were delivered with alternative drillings, Meccano fashion, in order to speed up erection.



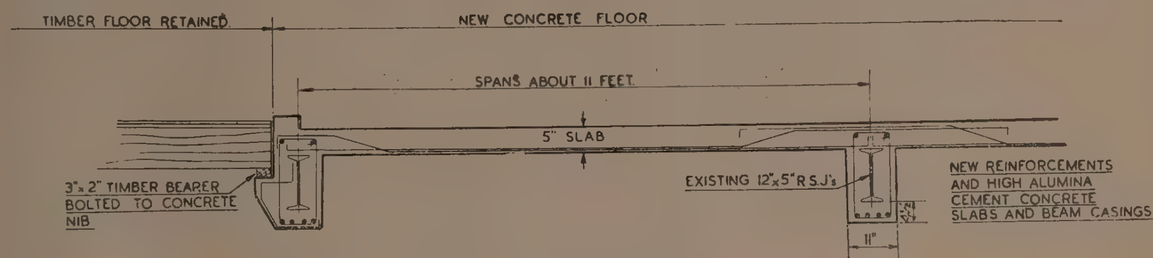
A. Old construction

Steel Beams

Parts of three of the floors in Warehouse "B" are now occupied by vats and kettles containing soya oil. The original timber secondary joists and floor boards would have been unsuitable in this vicinity, due to the risk of instantaneous combustion which arises when timber has absorbed soya oil either from spillage or vapour.

The timber flooring was therefore removed, but the steel main beams were retained to support new slabs of high alumina cement concrete. The steel beams alone were inadequate to carry the extra loads and were strengthened by the addition of rod reinforcements in the bottom of the concrete beam casings. This is shown in Fig. 5, which also indicates how the timber secondary joists in the adjoining bay were supported at the termination of the new concrete work.

The aggregate used for the beam casings was $\frac{3}{8}$ in.—$\frac{1}{4}$ in. and the beam shutters were vibrated to assist good compaction.



B. New construction

Fig. 5. Strengthening of Steel beams to carry new concrete floor

Where the ends of the timber main beams supported in walls were found to be rotten, the rotted end was cut away and the remainder of the beam supported by strengthening channels arranged as before but bearing at one end in the stone wall. A "stirrup" joist was provided at this end to carry the free end of the timber beam. See Fig. 4 (b). In some cases the one pair of strengthening channels served the dual function of dealing with rot and carrying additional loads from machines.

The timber secondary joists were augmented by new secondaries of timber or rolled steel sections as necessary, bearing on the strengthened timber-steel main beams.

The old brackets on the cast-iron columns were not able to take these extra loads, and the columns were therefore cased in concrete to octagonal section with eight longitudinal reinforcements and spiral binding. This casing was further required because the lowest suspended floor of this warehouse was removed, and the 12 in. diameter columns would have been without other assistance against buckling over a length of 17 feet with unbolted joints at their mid-heights.

Acknowledgement

The author is grateful to Messrs. Spillers, Ltd., for permission to publish these notes.

Book Review

Sound Insulation and Room Acoustics, by Per. V. Bruel, M.Sc., D.Sc. (Copenhagen). Translated by J. M. Borup. (London: Chapman & Hall.) 9 in. × 6 in. 275 pp. Illus. 35s.

Specialised academic investigation coupled with a widespread consultative practice in the acoustics field qualify Dr. Bruel to set down an authoritative survey of the principles and practice of modern acoustical theories.

This work is the result and in it the author treats comprehensively with the fundamental principles in the science of sound which affect acoustics of buildings and explains electrical methods for the measurement of sound, reverberation time and insulation values.

The work includes a useful survey in the field of physiological acoustics, including the frequency spectra of speech sounds and the peculiarities of the human ear as affecting the assessment of electrical measurements.

The more mathematical section on fields of sound leads to the exposition of reverberation formulæ and the section on absorbents gives complete and valuable information upon the control of the absorption-frequency characteristics based upon practical test.

The section on sound insulation and the damping of noise includes nomograms useful for the preliminary determination of insulation required to meet specific conditions and practical examples show how this can be achieved in practice.

The damping of noise is dealt with in a comprehensive manner, but the elimination of the transmission of structural vibration is only lightly touched upon.

The section on room acoustics includes a critical examination of the findings of all the previous well-known workers in this field, and on the whole the volume may be said to represent a masterly summary of the conclusions to be found in the 126 references enumerated in the work.

The translation is exact, and the illustrations and printing, clear and well presented.

The work is to be highly recommended to all interested in any way in the subject. C. W. G.

Correspondence

The Institution, whilst being at all times pleased to open its columns to correspondence, cannot accept responsibility for the opinions expressed.

The Analysis of Continuous Ridged Portal Frames

To the Editor of THE STRUCTURAL ENGINEER

Sir,—A method has recently been described[1] for analysing continuous ridged portal frames of any number of spans by calculating fixed-end moments and forces produced by the loading, and then imposing a succession of rotational and linear displacements at eaves joints until the restraining moments and forces at these points all become negligibly small.

An important factor affecting the usefulness of such a method of approximation to the true behaviour of the structure is the rate of convergence. For angular displacements the convergence is extremely rapid; in the usual case of symmetrical single-pitch roof members of constant inertia along the length the carry-over factor for moments is only —1/7 (as compared with the value of $+\frac{1}{2}$ encountered in the analysis of continuous beams), but for linear displacements the convergence is not so good, the carry-over factor for horizontal forces in the roof members being —1.

In order to speed up the process it was suggested that group displacements, representing horizontal displacements applied simultaneously to several joints, should be formed. In particular the "block" operation, representing equal displacements at each of the eaves joints, was found to be useful in reducing the net restraining force at eaves level to zero.

It would be an obvious advantage if group displacements were devised which could be used to eliminate the horizontal restraining force at any chosen eaves joint without affecting the forces at any other. In this way it would be possible to avoid carry-over of horizontal forces in the roof members while horizontal displacements were being applied, and it is the purpose of this note to show how this can be done.

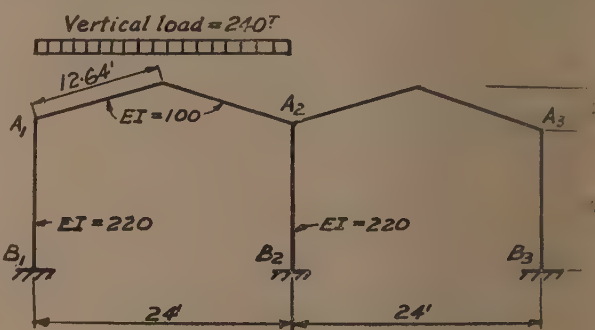
Consider the two-bay frame shown in Fig. 1. This frame and the first six lines of the operations table are taken from the previous article; the last three lines having been added to show the required group displacements. For instance, the symbol $\triangle G_1 = 1$ implies the three displacements $\triangle u_{A1} = 0.434$, $\triangle u_{A2} = 0.311$ and $\triangle u_{A3} = 0.255$ acting together, and these displacements produce a change in the horizontal force at A_1 of $+0.64$ while producing no change in forces at A_2 and A_3.

Fig. 2 shows how these group displacements have been used to solve for the deflexions and rotations at eaves level joints of the frame. Appropriate changes in G's and θ's are made in turn until the residual moments and forces are small at line (a). It will be seen that the displacements have been made in a systematic manner without recourse to intuition as to the final values expected, and the process has converged after only three cycles of displacements to G's and θ's. On line (b) values of θ's and G's have been added up, and u-values have been computed. For instance,

$$
\begin{aligned}
u_{A1} = & -318\times0.434 = -138.0 \\
& +365\times0.311 = +113.5 \\
& -\ 34\times0.255 = -\ \ 8.7 \\
& \qquad\qquad\qquad\ \ -\ 33.2
\end{aligned}
$$

Using these values of θ's and u's, residual moments and forces at A_1, A_2, and A_3 have been computed and entred along line (b). Small errors made up to this poit in the table are now apparent, and the procedure has een continued until at line (c) a satisfactory solution has been found.

It remains to show how the group operations vere devised, and to do this we consider the frame show in Fig. 3. Taking the joints A_1, A_2 and A_3 *to be restrined against rotation but free to sway*, it is our intentio to



Operations Table

	ΔM_{A1}	ΔH_{A1}	ΔM_{A2}	ΔH_{A2}	ΔM_{A3}	ΔH
$\Delta\theta_{A1} = 1$	+82·7	+0·77	−3·96	−5·93	0	0
$\Delta\theta_{A2} = 1$	−3·96	−5·93	+110·4	+6·70	−3·96	−5·
$\Delta\theta_{A3} = 1$	0	0	−3·96	−5·93	+82·7	+0
$\Delta u_{A1} = 1$	+0·77	+3·60	−5·93	−2·96	0	0
$\Delta u_{A2} = 1$	−5·93	−2·96	+6·70	+6·56	−5·93	−2·
$\Delta u_{A3} = 1$	0	0	−5·93	−2·96	+0·77	+3·
$\Delta G_1 = 1$, $\Delta u_{A1} = 0·434$, $\Delta u_{A2} = 0·311$, $\Delta u_{A3} = 0·255$	−1·51	+0·64	−2·00	0	−1·65	0
$\Delta G_2 = 1$, $\Delta u_{A1} = 0·311$, $\Delta u_{A2} = 0·378$, $\Delta u_{A3} = 0·311$	−2·00	0	−1·16	+0·64	−2·00	0
$\Delta G_3 = 1$, $\Delta u_{A1} = 0·255$, $\Delta u_{A2} = 0·311$, $\Delta u_{A3} = 0·434$	−1·65	0	−2·00	0	−1·51	+0·

Fig. 1

find what displacements will be produced at A_1, A and A_3 by a unit load applied at A_1 as shown. The m is to replace the ridged bays successively from th by a single member. Defining the sway stiffness any

member of the frame as the force required to produce unit horizontal movement without rotation at one end while the other end is fixed, roof member A_2A_3 has a stiffness of 2.96 and column A_3B_3 has a sway stiffness of 0.64, so they may be replaced by a single vertical member C_3D_3 connected to A_2 by a rigid arm A_2C_3 and of stiffness S given by

$$\frac{1}{S} = \frac{1}{2.96} + \frac{1}{0.64}$$

$$\text{or } S = 0.526$$

A_2B_2 and C_3D_3 have a combined sway stiffness of $0.64 + 0.526 = 1.166$ so that A_1A_2, A_2B_2 and C_3D_3 may be replaced by a single vertical member C_2D_2, the stiffness of which is given by

$$\frac{1}{S} = \frac{1}{2.96} + \frac{1}{1.166}$$

$$\text{or } S = 0.836.$$

Having reduced the frame to the two members A_1B_1 and C_2D_2 as far as sway displacements are concerned, it is now possible to apportion the applied load between A_1B_1 and the rest of the frame in the ratio 0.64 to 0.836, i.e., the horizontal force at B_1 is 0.434. Dividing the remainder, 0.566, between A_2B_2 and C_3D_3 in the ratio 0.64 to 0.526 the horizontal force at B_2 is found to be 0.311, and at D_3 (or B_3), 0.255.

The horizontal forces produced at B_1B_2 and B_3 by a unit horizontal load at A_1, when A_1A_2 and A_3 are free to sway but are not free to rotate, are therefore 0.434, 0.311 and 0.255 respectively, and since the three stanchions have equal stiffness, the horizontal displacements at A_1A_2 and A_3 are also in this ratio. Putting $\triangle u_{A1} = 0.434$, $\triangle u_{A2} = 0.311$ and $\triangle u_{A3} = 0.255$ in the operations table, we find that

$$\begin{aligned} \triangle M_{A1} = \quad & 0.434 \times +0.77 = +0.33 \\ & 0.311 \times -5.93 = -1.84 \\ & \overline{\qquad\qquad -1.51} \end{aligned}$$

and so on, and incidentally verify that $\triangle H_{A2} = \triangle H_{A3} = 0$, which shows that the group has been correctly formed.

The lower part of Fig. 3 shows corresponding calculations for a force applied at A_2.

Relaxation Table

	M_{A1}	H_{A1}	M_{A2}	H_{A2}	M_{A3}	H_{A3}
θ's = u's = 1	−120	+180	+120	−180	0	0
ΔG_1 = −30	+333	−12	+720	−180	+495	0
ΔG_2 = +30	−267	−12	+372	+12	−105	0
$\Delta\theta_{A1}$ = +3·	−2	−9.5	+359	−7.0	−105	0
$\Delta\theta_{A2}$ = −3·	+11	+10.1	−5	−29.1	−92	+19.6
$\Delta\theta_{A3}$ = +1·1	+11	+10.1	−9	−35.6	−1	+20.4
ΔG_1 = −2	+41	−2.7	+31	−35.6	+32	+20.4
ΔG_2 = +6	−79	−2.7	−39	+2.8	−88	+20.4
ΔG_3 = −3	−29	−2.7	+21	+2.8	−43	+1.2
$\Delta\theta_{A1}$ = +0.3	−4	−2.5	+20	+1.0	−43	+1.2
$\Delta\theta_{A2}$ = −0.2	−3	−1.3	−2	−0.3	−42	+2.4
$\Delta\theta_{A3}$ = +0.5	−3	−1.3	−4	−3.3	−1	+2.8
ΔG_1 = +2	−6	0	−8	−3.3	−4	+2.8
ΔG_2 = +5	−16	0	−14	−0.1	−14	+2.8
ΔG_3 = −4	−9	0	−6	−0.1	−8	+0.2
$\Delta\theta_{A1}$ = +0·	−1	+0.1	−6	−0.2	−8	+0.2
$\Delta\theta_{A2}$ = +0·	−1	−0.2	0	+0.1	−8	−0.1
$\Delta\theta_{A3}$ = +0	−1	−0.2	0	0	0	0
θ_{A1}=3.6 G_1=−318 u=−33.2 θ_{A2}=−3.45 G_2=365 u=28.5 θ_{A3}=+1.7 G_3=−34 u=17.6	−3	−0.7	+2	−1.4	−1	+0.7
ΔG_1 = +1	−5	−0.1	0	−1.4	−3	+0.7
ΔG_2 = +2	−9	−0.1	−2	−0.1	−7	+0.7
ΔG_3 = −1	−7	−0.1	0	−0.1	−5	+0.1
$\Delta\theta_{A1}$ = +0·	+1	0	0	−0.7	−5	+0.1
$\Delta\theta_{A3}$ = +0·	+1	0	0	−1.0	−1	+0.1
ΔG_2 = +2	−3	0	−2	+0.3	−5	+0.1
$\Delta\theta_{A1}$ = +0·	0	0	−2	+0.1	−5	+0.1
$\Delta\theta_{A2}$ = +0·	0	−0.1	0	+0.2	−5	0
$\Delta\theta_{A3}$ = +0·	0	−0.1	0	−0.1	0	0
θ_{A1}=3.74 G_1=−317 u=−31.7 θ_{A2}=−3.43 G_2=369 u=30.0 θ_{A3}=1.81 G_3=−35 u=19.7	+1	+0.2	−2	−0.4	0	+0.2

Fig. 2

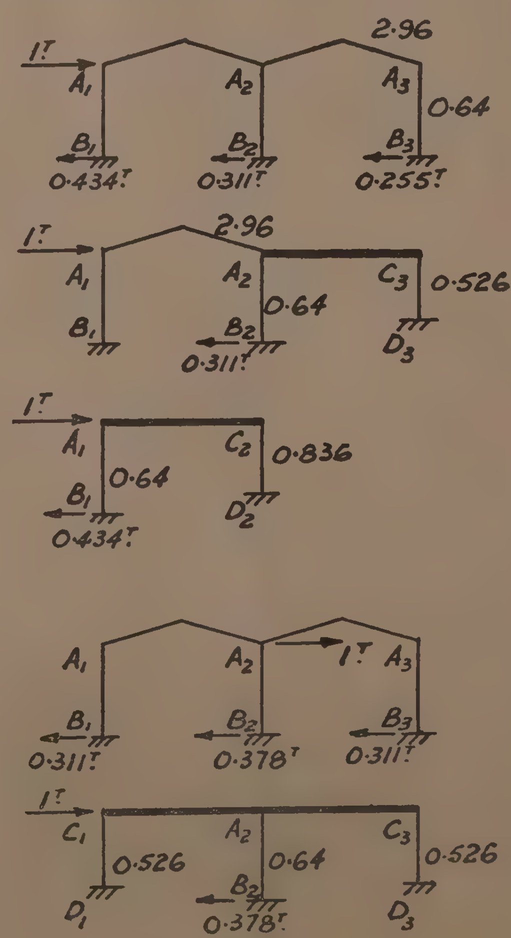


Fig. 3

For frames with more spans it is clearly possible to use the same method to obtain the required groups. The advantage of working with these groups is that carry-over of forces from end to end of the roof members is eliminated when imposing horizontal displacements, so that operations can be made in a quite automatic manner without calling on the computer's intuition to make the process converge rapidly. A disadvantage lies in the work required to form the groups, and an additional step is required in checking the solution, as values of u's must be calculated from G-values before residual M's and H's can be found. Nevertheless the groups should prove useful where a frame of several bays is to be analysed for a succession of load conditions, and the preliminary work becomes inconsequential.

Yours, etc.,
E. Markland
(Associate-Member).

Nottingham.
June 12th, 1952.

[1]Markland, E. The analysis of continuous ridged portal frames. THE STRUCTURAL ENGINEER, Vol. 30, No. 5, p. 101 (May, 1952).

To the Editor of THE STRUCTURAL ENGINEER

Sir,—In his paper, "The Analysis of Continuous Ridged Portal Frames," published in THE STRUCTURAL ENGINEER, May, 1952, Mr. E. Markland suggested that the span of models of multi-bay frames of this type could be reduced provided that the ratio $\dfrac{I}{L}$ for the symmetrical roof member remained constant and the rise h remained unaltered.

This method has been used in the model analysis of an encastre four-bay ridged portal frame (Fig. 1) in the Department of Civil and Mechanical Engineering at the University of Nottingham. The indirect large displacement method used by previous investigators[1] and [2] was adopted, the models being made from $\frac{1}{8}$ in. sheet of I.C.I. "Acrylic" Perspex. A model scale of $\frac{1}{2}$ in. to one foot was chosen but without reduction of the span this scale would have necessitated an unwieldy model, 48 in. long, so the span was reduced to one-third of the geometric scale, Fig. 2. In order that the model should be as flexible as possible the roof member was made 0.175 in. deep, the stanchion depth corresponding to a second moment of area ratio of 1 to 2.2 being 0.298 in.

The extensions at the feet of the model were nailed to a drawing-board covered with glossy paper and appropriate deformations introduced at the feet to

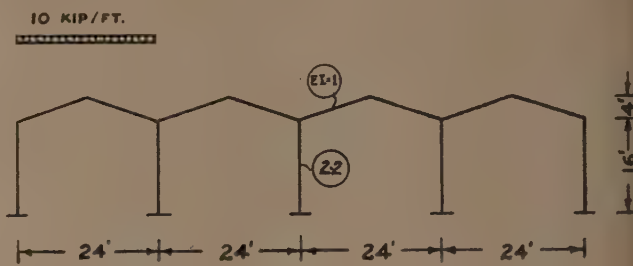


Fig 1

obtain the influence lines for moment, horizontal and vertical thrusts due to a uniformly distributed vertical load over the left-hand span. The distorted shape of this span was pricked on to the underlying paper by running a needle down fine grooves made in the edge of the roof member. The influence line ordinates were then measured in the direction of action of the load with the aid of a hand lens and inch scale graduated in 1/100ths.

The effect on the stanchions of a roof member of distorted dimensions will be exactly similar to the effect of the original roof member provided that the same fixed

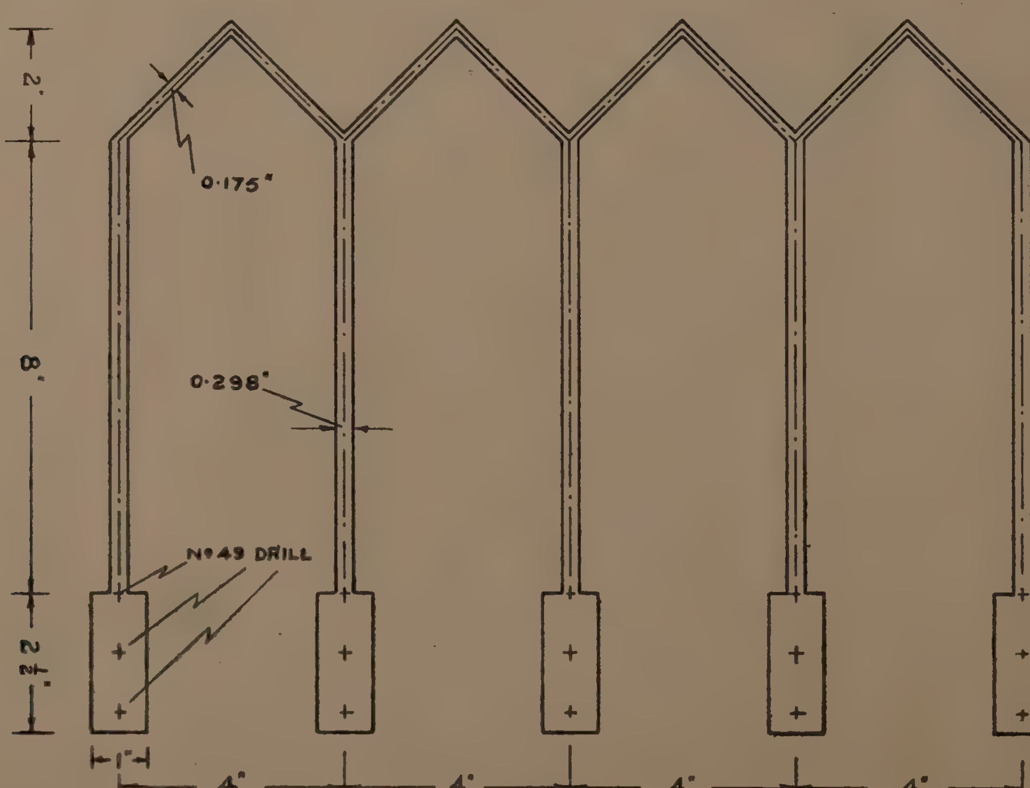


Fig. 2

nd moments, horizontal and vertical thrusts act in the oof member. In other words, the sway and rotation of he eaves joints will be the same if the distorted roof nember is loaded in a manner which will produce the ame fixed end constraints at the eaves as the un-listorted roof member. Suppose that the span of the listorted roof member is n times the span of the true oof member and both are loaded with the same load per unit length, the fixed end moments, horizontal and vertical thrusts are respectively n^2, n^2 and n times those of the true roof member. As the stanchions are unchanged, the constraints at the feet will be modified by the same factors. Therefore, if the distorted shape of an maginary full-sized frame of reduced span is found from the model and the constraints at the feet of the stanchions determined for the same system of loading as for the true full-sized structure, then the moment, horizontal and vertical thrusts at the feet of the true structure may be determined by dividing by n^2, n^2 and n respectively.

If the applied load were not vertical then it would be necessary to resolve the load into its horizontal and vertical components. The vertical component must be corrected for the change in span but no correction would be necessary for the horizontal component as the fixed end moments and thrusts due to this would be the same, the rise of the span remaining unchanged. Due to the scale effect an additional correction must be applied whenever moment is being determined.[3]

applied load leading to small constraints at the fixed ends remote from the loaded span.

On summing the horizontal and vertical reactions little error is found but considerable error is evident in the moments at the eaves joints, for in several cases the statically unbalanced moment is of a similar order to the moments themselves. The method of calculation of these moments was to consider the statical equilibrium about each of the members at the eaves joints which meant that small errors in thrusts at the feet, especially vertical thrust, would have serious effects on these moments, e.g., a variation of 0.5 kip. in the vertical thrust at B_5, would lead to a change of 36 kip. ft. in the moment in the roof member to the right of A_2.

While the assumption of a uniformly distributed load on only one span involves least experimental work, as only one influence line has to be plotted for each test, it represents the worst case for the derivation of accurate results. A reduction of approximately 0.25 kip in the vertical thrust at B_5 would make the eaves joints statically correct, but 0.25 kip. is small in comparison to the vertical load carried by the frame.

Yours, etc.,

J. Springfield.

Nottingham.
June 24th, 1952.

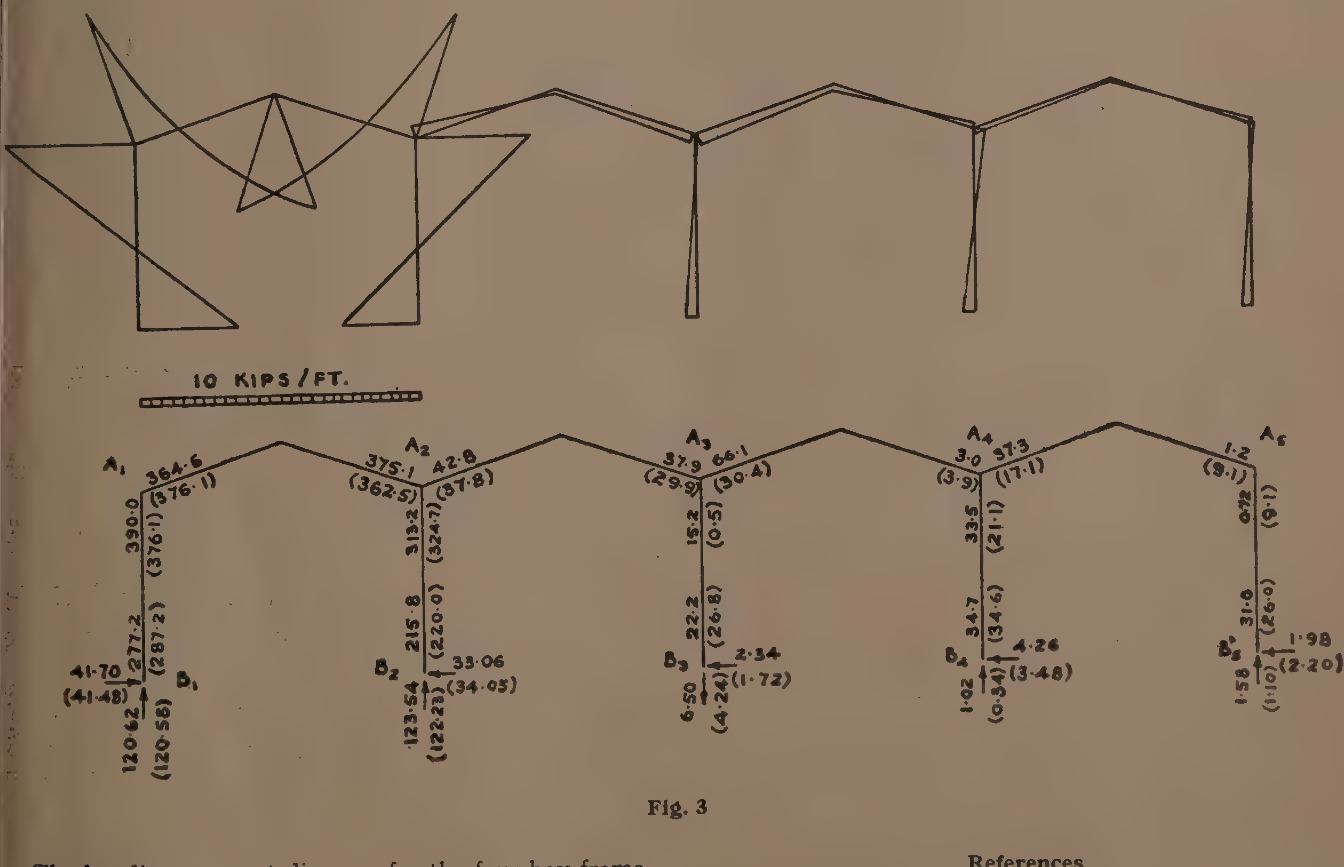


Fig. 3

The bending moment diagram for the four bay frame is shown in Fig. 3, the experimentally determined moments at each joint being given in the accompanying line diagram, followed by the calculated value shown in brackets (determined by the relaxation method due to Markland). Comparison of the results shows a progressive increase in the proportionate difference between the two values as the distance from the loaded span increases. This is due to a reduction in the effect of the

References

[1] "Simple Experimental Solutions of Certain Structural Design Problems." A. J. S. Pippard and S. R. Sparkes. JOURNAL I.C.E. No. 1, 1936-37.

[2] "Structural Analysis by Models," P. S. Pell, N. E. Thompson, and R. C. Coates. CIVIL ENGINEERING AND PUBLIC WORKS REVIEW. Vol. 47, No. 553. July, 1952.

[3] "The Analysis of Engineering Structures," A. J. S. Pippard, and J. F. Baker. Arnold, London, p 559.

Also the writer's thesis "The Model Analysis of Structures" presented to the University of Nottingham, May, 1952.

Institution Notices and Proceedings

PRESIDENTIAL ADDRESS—SESSION 1952-53

A General Meeting of the Institution of Structural Engineers will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 9th, 1952, at 6 p.m., when Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E., will be installed as President for the Session 1952-1953, and will give the Presidential Address.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, October 23rd, 1952

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. A. J. Harris, B.Sc.(Eng.), A.M.I.C.E., will give a paper on "Hangars at London Airport—Design of Large Span Prestressed Concrete Beams."

Thursday, November 13th, 1952

Ordinary Meeting, 6 p.m., when the MacLachlan Lecture will be given by Mr. L. E. Ward, A.M.I.Struct.E. The title of the Lecture will be "The Design and Construction of a Three-Bay Aluminium Aircraft Hangar at London Airport."

Thursday, November 27th, 1952

Ordinary General Meeting for the election of members at 5.55 p.m., followed by a Joint Meeting with the British Section of the Société des Ingénieurs Civils de France, at 6 p.m., when M. Léviant will give a paper entitled "An Introduction to Vacuum Concrete."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1953

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on January 6th and 7th, 1953 (Graduateship) and January 8th and 9th (Associate-Membership).

REPRESENTATION

The Council have made the following nominations of members to represent the Institution :—

L.C.C. School of Building—Governing Body
Mr. T. P. Sturdee (Member) (renominated).

Regional Advisory Council for Higher Technological Education, London and Home Counties
Mr. L. E. Kent, B.Sc.(Eng.), M.I.C.E. (Vice-President) (renominated).

City of Sheffield Education Committee : Building Advisory Committee
Professor J. Husband, F.R.C.Sc.I., M.I.C.E. (Past-President).

MACLACHLAN LECTURE, 1952

The Council announce that as a result of the Competition held for the MacLachlan Lecture, 1952, the award has been made to Mr. L. E. Ward (Associate-Member), for his lecture entitled "The Design and Construction of a Three-Bay Aluminium Aircraft Hangar at London Airport," which will be presented at a meeting of the Institution on Thursday, November 13th, 1952.

MACLACHLAN LECTURE COMPETITION, 1953

The closing date for the receipt of entries for the next MacLachlan Lecture Competition will be Tuesday, March 31st, 1953. Particulars of the Competition are as follows :—

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture, and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering so long as in every second year the subject shall be confined to steel structures. (1953 is one of these years.)

3. Entrance into the competition for the Lecture shall be confined to Associate-Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s. 0d.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture is submitted or because no lecture submitted is considered to be of sufficient merit to warrant an award, or for any other reason, the Institution shall transfer these sums to the Research Fund of the Institution.

Particulars of the Competition for 1953

1. The MacLachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1953.

2. The subject of the Lecture shall be confined to steel structures.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Tuesday, March 31st, 1953.

RESEARCH AWARDS

The Council have instituted a Research Prize Fund from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches

and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :—

(a) investigations of an experimental or analytical character ;

(b) studies of historical or statistical records ;

(c) improvements in principles or methods of construction ;

(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;

(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms :— A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :—

(a) the nature of the subject and its conclusions ;

(b) the value of the paper in advancing the science and art of structural engineering ;

(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1951, and September, 1952, is October 31st, 1952.

DRURY MEDAL AWARD

The fourth competition for the above award will take place in 1953. The subject is the design of the structure of a new factory building. The material of construction is entirely at the choice of the competitor. The competition has been designed to encourage ingenuity of structural arrangement. Economy in the use of steel is an important feature of this year's competition.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary ; envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st, 1953.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

HONOURS AND AWARDS

In offering their sincere congratulations to the following member on the distinction recently conferred upon him, the Council feel that they are also expressing the good wishes of the Institution.

ORDER OF THE BRITISH EMPIRE—M.B.E.
S/Ldr. H. Jennings (Associate-Member).

COMMUNICATIONS TO MEMBERS

The Institution is experiencing difficulty in receiving replies to communications addressed to the undernoted members, and the Secretary would be glad to be notified of their present addresses as soon as possible :—

W. S. BLOUNT, P. M. GOUGH, W. C. KEEN (Associate-Members).
G. CONNELLY, J. C. KERR, N. C. SAGAR (Graduates).

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will be a visit to the steel mills of Messrs. Dorman Long & Co., Ltd., at Middlesbrough. The party will travel to Middlesbrough by coach on the night of Thursday, October 9th, visit the mills on the 10th, and return on Saturday, October 11th. All accommodation and meals are being arranged.

Hon. Secretary : C. Allen Brown, 43, Coolgardie Avenue, Highams Park, London, E.4.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Tuesday, October 7th, 1952

Chairman's Address, by Mr. W. Bates (Member), followed by a film on " The Erection of the Rainbow Bridge at Niagara Falls." Visit of the President and the Secretary of the Institution. The meeting will be held at the University of Manchester at 6.30 p.m.

Wednesday, October 29th, 1952

Three Short Lectures :—

Mr. John Drinkwater (Graduate), on " Reinforced Concrete Structures in Coke Oven Plants."

Mr. J. W. White, A.M.I.C.E. (Associate-Member), on " Some Useful Graphs for the Design of Foundations."

Mr. R. W. Williams, on " Secondary Effects of Details on the Stresses in Structural Steelwork."

The meeting will be held in the Reynolds Hall, College of Technology, Manchester, at 6.30 p.m., preceded by tea at 5.45 p.m.

Wednesday, November 19th, 1952

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator), on " Unusual Design for a Large Constructional Shop," at the College of Technology, Manchester, 6.30 p.m.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 24, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Saturday, October 11th, 1952

Annual Dinner, at the Botanical Gardens, Birmingham.

Friday, October 24th, 1952

Chairman's Address, by Mr. H. J. Morris, M.B.E., A.M.I.C.E. (Member), at the James Watt Memorial Institute, Birmingham, 6 p.m.

The President and the Secretary of the Institution will attend both the above meetings.

Tuesday, November 18th, 1952

Film : "Fawley Refinery," Parts 1 and 2, at Derby, 7 p.m.

Friday, November 28th, 1952

Mr. E. McMinn, on "Tubular Structures," at the James Watt Memorial Institute, 6 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Thursday, October 30th, 1952

Meeting at the James Watt Memorial Institute, Birmingham, 7 p.m. Details to be announced.

Wednesday, November 26th, 1952

Joint Meeting with the Student Section of the Midlands Association of The Institution of Civil Engineers. Mr. E. T. Edwards, F.G.S., A.M.I.Mech.E., on "Boreholes in the Midlands for Water Supply and Foundations," at Birmingham Civic Centre (Room 129), 6.30 p.m.

Hon. Secretary : F. G. Fletcher, 60, Brean Avenue, South Yardley, Birmingham, 26.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, October 14th, 1952

Chairman's Address, by Mr. A. V. Buttress (Member), at Middlesbrough.

Wednesday, October 15th, 1952

The above meeting will be repeated at Newcastle. The President and the Secretary of the Institution will attend on both occasions.

Tuesday, November 4th, 1952

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator), on "Unusual Design for a Large Constructional Shop," at Middlesbrough.

Wednesday, November 5th, 1952

The above meeting will be repeated at Newcastle.

Tuesday, December 2nd, 1952

Mr. D. M. Brotton, B.Sc., Ph.D. (Graduate), on "Relaxation Methods," at Middlesbrough.

Wednesday, December 3rd, 1952

The above meeting will be repeated at Newcastle.

All meetings will commence at 6.30 p.m., the Middlesbrough meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and the Newcastle meetings in the Neville Hall, near the Central Station.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, October 7th, 1952

Chairman's Address, by Mr. M. C. Gillies (Member).

Tuesday, November 4th, 1952

Mr. Stanley Marchant, B.Sc., A.M.I.Mech.E., on "Theory and Practice of Prestressed Concrete."

Tuesday, December 9th, 1952

Films—"Welded Structures" and "New Tyne Bridge"—kindly lent by Messrs. Dorman Long & Co., Ltd.

All meetings will be held at the College of Technology, Belfast, at 6.45 p.m., preceded by tea at the Overseas League Premises, Wellington Place, Belfast, at 6 p.m.

Hon. Secretary : S. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Monday, October 27th, 1952

Mr. M. Bridgewater, on "Aluminium in Structural Engineering," at the Ca'doro Restaurant, Glasgow, 6 p.m.

Tuesday, October 28th, 1952

Annual Dinner and Dance, at the Grosvenor Restaurant, Glasgow, at 6 o'clock for 6.30 p.m.

The President and the Secretary of the Institution will be present on both the above occasions.

Tuesday, November 18th, 1952

Mr. E. McMinn, on "Tubular Structures," at the Ca'doro Restaurant, Glasgow, 6 p.m.

Tuesday, December 16th, 1952

Mr. J. H. Huntley, on "Structural Design of Cranes," at the Ca'doro Restaurant, Glasgow, 6 p.m.

Hon. Secretary : D. G. Drummond, B.Sc., M.I.Struct.E., A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held at the Duke of Cornwall Hotel, Millbay, Plymouth, on Wednesday, November 5th, 1952, at 7 p.m. The President and the Secretary of the Institution will attend the meeting.

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10-12, Market Street, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Friday, October 17th, 1952

Chairman's Address, by Colonel R. D. Heseltine, T.D., D.L., M.I.Struct.E., at Cardiff. The meeting will be attended by the President and the Secretary of the Institution.

Wednesday, October 29th, 1952

The Chairman's Address will be repeated at Swansea, and will be followed by discussion.

Saturday, November 1st, 1952

The Chairman's Address will be repeated at Colwyn Bay and will be followed by discussion.

Thursday, November 13th, 1952

Mr. R. G. Braithwaite, M.I.C.E., on "Electric Screw Piling," at Cardiff.

Wednesday, November 19th, 1952

The above meeting will be repeated at Swansea.

Monday, December 1st, 1952

A meeting will be held at Cardiff, at which films will be shown.

Wednesday, December 3rd, 1952

A meeting will be held at Swansea, at which films will be shown.

Meetings in Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel, at 6.30 p.m.

Meetings at Colwyn Bay will be held at the County Buildings, at 6 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, October 17th, 1952

Chairman's Address by Mr. E. N. Underwood, B.Sc., M.I.C.E. (Member). The subject of the Address will be "Problems in Practice." The meeting will be attended by the President and the Secretary of the Institution.

Friday, November 7th, 1952

Combined meeting with the Institution of Civil Engineers. Mr. G. P. Bridges, A.M.I.C.E., L.R.I.B.A. (Member), on "The Design and Construction of Reinforced Concrete Silos and Bunkers."

Friday, December 5th, 1952

Mr. P. J. Ward on "The Design and Erection of Television Masts."

All meetings will be held in the University of Bristol Geology Lecture Theatre at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 39, Effingham Road, St. Andrew's Park, Bristol, 6.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Thursday, October 30th, 1952

Chairman's Address by Mr. D. R. S. Wilson (Member). The meeting will be attended by the President and the Secretary of the Institution.

Wednesday, November 19th, 1952

Mr. G. C. Cummings, B.Sc., on "Concrete Grain Silos at Louth."

Wednesday, December 17th, 1952

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator), on "Unusual Design for a Large Constructional Shop."

All meetings will be held at the University, Leeds, at 6.30 p.m.

Hon. Secrctary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

INSTITUTION LIBRARY

The following volumes have been added to the Library :—

ALLIN, Russell V. *Resistance of Piles to Penetration.* London, 1951.

BRUEL, Per V. *Sound Insulation and Room Acoustics.* London, 1951. Presented by Mr. C. W. Glover.

BRUNOLI, C. L. *Telai Elastici.* Milan, 1951.

CHALMERS, Bruce (Editor). *Progress in Metal Physics, 2.* London, 1950.

CLARK, D. A. R. *Advanced Strength of Materials.* London and Glasgow, 1951. Presented by Dr. A. A. Fordham.

CRAFTS, W. and LAMONT, J. L. *Hardenability and Steel Selection.* London, 1949. Presented by Mr. G. S. Gowland.

CRESSWELL, W. T., revised by T. R. D. Davies. *The Law Relating to Building and Engineering Contracts,* 5th Edition. London, 1952.

DALZELL, J. R. and TOWNSEND, G. *Concrete Block Construction,* Chicago and London, 1951. Presented by Mr. C. W. Glover.

DAWNAYS, Ltd. *Structural Steel Handbook.* London, 1951. Presented by the Publishers.

FABER, O. and CHILDE, H. L. *The Concrete Year Book,* 1952. London, 1952.

FRASER, M. *Work of the Singapore Improvement Trust,* 1950. Singapore, 1951. Presented by the Publishers.

FREUDENTHAL, A. M. *The Inelastic Behaviour of Engineering Materials and Structures.* New York and London, 1950. Presented by Mr. W. Basil Scott.

GEDDES, Spence. *Estimating for Building and Civil Engineering Works.* London, 1951.

GUYON, Y. *Beton Precontraint : Etude Theorique et Experimentale.* Paris, 1951. Presented by Dr. K. Hajnal-Konyi.

HERMANN, G. *Experimentelle Untersuchung der Spannungsverteilung in Platten von Streifenundamenten. Theoretische Untersuchungen uber die Zentrischer Einzellast.* Zurich, 1950.

HOLLOWOOD, B. *Cornish Engineers.* Camborne, England, 1951. Presented by Mr. S. J. Crispin.

HUNTER, L. E. *Construction with Moving Forms.* London, 1951. Presented by Mr. Cyril Parry.

LACKNER, E. *Berechnung mehrfach gestutzter Spundwande.* Berlin, 1950. Presented by Dr. G. G. Meyerhof.

LEEMING, J. J. *Road Curvature and Superelevation.* London, 1951. Presented by Mr. D. M. O'Herlihy.

MALLET, Ch. and PACQUANT, J. *Les Barrages en Terre.* Paris, 1951. Presented by Mr. P. J. Gerrard.

MANNING, G. P. *The Displacement Method of Frame Analysis.* London, 1952. Presented by Dr. E. H. Bateman.

MARSHALL, W. T. *The Fundamental Principles of Reinforced Concrete Design.* London and Glasgow, 1951. Presented by Mr. A. F. Holt.

MERCER, L. Boyd. *Law of Grading for Concrete Aggregates.* Melbourne, 1951. Presented by Mr. R. V. Chate.

MOORE, N. P. W. (Editor). *The Practical Engineer's Pocket Book.* London, 1952.

OLSEN, G. A. *Strength of Materials.* London, 1951. Presented by Mr. E. Markland.

PARKER, H. *Simplified Mechanics and Strength of Materials.* New York and London, 1951. Presented by Mr. Newman Tate.

PRENTIS, E. A. and WHITE, L. *Underpinning: Its Practice and Applications.* 2nd Edition. New York, 1950. Presented by Mr. J. L. M. Uren.

PROBST, E. H. and COMRIE, J. *Civil Engineering Reference Book.* London, 1951.

SEELYE, E. E. *Data Book for Civil Engineers*, Vols. I and II. New York and London, 1951. 2nd Edition.

SKAYANNIS, A. P. *System of Tables for Quick and Accurate Solving of any Continuous Beam.* London, 1949. Presented by the Author.

STEWART, D. A. *The Design and Placing of High Quality Concrete.* London, 1951. Presented by Mr. P. B. R. Johnson.

SUTHERLAND, H. and BOWMAN, H. L. *Structural Theory.* 4th Edition. New York and London, 1950.

THIRLWELL, J. B. *Strength of Materials.* London, 1952. Presented by the Publishers.

TOFT, L. and MCKAY, A. D. D. *Practical Mathematics*, Vol. I. 3rd Edition. London, 1951.

TOFT, L. *Definitions and Formulæ for Students: Practical Mathematics.* 3rd Edition. London, 1951.

WEST, E. G. *The Welding of Non-Ferrous Metals.* London, 1951. Presented by Mr. S. M. Reisser.

WOOD, R. D. *Principles of Quantity Surveying.* London, 1951. 2nd Edition.

YARNELL, J. *Resistance Strain Gauges, their Construction and use.* London, 1951. Presented by Mr. H. G. Foster.

YOUNG, J. McHardy. *Structural Theory and Design*, Vol. II. London, 1951. Presented by Mr. D. T. Williams.

Aluminium Development Association. *Proceedings at Symposium on Welding and Riveting Larger Aluminium Structures.* London, 1951. Presented by the Association.

Imperial Chemical Industries. *Codes and Regulations. Railways and Haulages and Road Transport of Engineering Plant and Materials. Safety Precautions.* London, 1951. Presented by the Publishers.

Institute of Physics. *Memorandum on Gamma-Ray Sources of Radiography.* London, 1952. Presented by the Institute.

Iron and Steel Institute. *Special Report No. 44. Surface Defects in Ingots and Their Products.* (Recommended Definitions.) London, 1951. Presented by the Institute.

Ninth International Road Congress, Lisbon, 1951. *Symposium of Papers. British Manufactured Mechanical Equipment for Road Construction and Maintenance.* London, 1951. Presented by Mr. P. M. Otway.

Second International Conference on Soil Mechanics and Foundation Engineering. *Proceedings.* Vols. I-IV. Rotterdam, 1948.

Engineering Society of Hong Kong. *Proceedings.* Vol. IV, 1950-1951. Hong Kong, 1951. Presented by Professor K. Billig.

International Geological Congress, Great Britain, 1948. *Report of the 18th Session.* Parts I, II, III, IV, VI, VIII, IX, XI, XIV, and XV. London, 1950 and 1951. Presented by Mr. Leslie Turner.

Roads and Road Construction. *Year Book and Directory*, 1950-51. London, 1951.

The Quebec Bridge. *Final Report of the Government Board of Engineers*, 1918. Vols. I and II. Ottawa, 1919. Presented by Mr. S. R. Banks.

The following papers have been accepted by the Literature Committee for filing in the Library and are available for reference :—

"The Assumed Deflection Method for the Determination of Transverse Stresses in Slabs supported on Two Sides," by Ronald Noble. (A precis of this paper was published in the August issue of the Journal.)

"Contributions to the Theory of Girder Walls," by H. L. B. Uhlmann. (A precis of this paper was published in the August issue of the Journal.)

"Structural Design of the Mediæval Cathedral," by Ronald Oates.

The paper deals with the development and building of the mediæval cathedral, and traces the evolution of structural design from the 9th to the 15th centuries. Mention is made of the development of the arch, the vaulted roof and the flying buttress, and the growth from the early Norman structures to the completely buttressed and vaulted Gothic cathedral. Several examples are given of the difficulties encountered, both in the building and in the maintenance of various cathedrals.

"Analysis of Vierendeel Trusses by the Automatic Method of Successive Corrections," by Jayme Ferreira da Silva, Junr.

"Routine Computation of Continuous Bridge Structures," by P. M. Tezner.

Book Reviews

Strain Gauges—Theory and Application, by Prof. Ir. J. J. Koch, Ir. R. G. Boiten, Ir. A. L. Biermasz, G. P. Roszbach, G. W. Van Santen. (London: Cleaver Hume Press, 1952. 95 pp., $8\frac{3}{4}$ in. × 6 in. 15s.)

This book contains six chapters on the various aspects of the use of electric resistance strain gauges by six different authors, each an expert in his own particular field.

The first chapter begins with a general introduction to strain gauge work. The author goes on to describe with the aid of very clear diagrams the components and construction of gauges, concluding with an account of their manufacture.

G. P. Roszbach gives a comprehensive survey of the various types of instruments used and the results which may be expected from them.

The most critical operation, namely, the technique of cementing the gauges to metallic test pieces is admirably covered by Ir. R. G. Boiten. It is, however, disappointing that no information is given about strain gauge measurements on concrete structures.

Chapter 4 concerns the evaluation and interpretation of the results; the author examines the influence on the results of variations in gauge factor, temperature, humidity, etc.

Prof. Dr. Ir. J. J. Koch deals capably with the mathematical theory involved in converting the strain readings into values of principal and shear stresses, he also summarises the theories of failure by Lamé, Guest and Huber v. Mises Hencky.

The final chapter gives interesting details of the use of strain gauges in various types of measuring instruments such as pressure gauges and dynamometers.

The diagrams and presentation are excellent, making the book a valuable asset to engineers requiring stress distributions in structural components. D. M. B.

Definitions and Formulae for Students: Practical Mathematics, by Louis Toft. (London: Pitman, 1951.) 38 pp., $5\frac{1}{2}$ in. × $4\frac{1}{4}$ in. 1s. 6d.

A third edition of this small booklet, which provides a useful collection of mathematical rules and formulæ, has been published.

Introduction to the Vacuum Concrete Processes*

By I. Leviant

I.—Historical

We have become accustomed to the pressure of the atmosphere in which we live and fail generally to realise the latent power it represents. But we have a simple means of recalling it to mind. If we consider a mountain lake, its waters also represent potential energy. To release this energy, all that need be done is to install a suitable plant (a hydro-electric station) at a lower point, where the waters of the lake exercise considerable pressures which can be utilised in work. In the same way, to "awake" atmospheric energy, it suffices to provide a source of low pressure (a vacuum unit) in relation to which the atmosphere acts as an infinite reservoir of compressed air.

The first demonstration of this was given three centuries ago: in effect, in 1652 Von Guericke formed the idea, which he realised shortly afterwards, of proving the existence and at the same time the power of atmospheric pressure by the celebrated "Magdeburg hemispheres" experiment.

Many years were to pass before this fertile idea found its field of application. It was to Karl P. Billner, of Philadelphia, that fell the honour of having introduced the use of atmospheric energy into constructional technique and in particular in view of the treatment of fresh concrete. The first time this treatment was presented to the public and the Press at Yale University, what struck the journalists was the presence side-by-side of a vacuum pump and of concrete; on the following day the caption appeared in the newspapers, "Vacuum Concrete," and the name—a little misleading, perhaps—was born.

II.—Analysis of the Vacuum Treatment

A simple illustration will serve better than theoretical explanation. At the moment this is being written, the concrete of a large building in Geneva is being poured. The pillars (columns 1 ft. × 1ft. × 9 ft.) are being treated by vacuum; for this purpose, the inner face of the shuttering is covered by a very simple filter (fabric and wire mesh). As soon as the concrete is filled in with the use of vibration, the filter is connected to a vacuum unit. The concrete is immediately compacted (its upper surface sinks) under the influence of the pressure of the atmosphere acting directly on the concrete without any machine or mechanical device. This compaction, or compacting process, is accompanied by appreciable expulsion of water through the filter and is such that after 15 minutes the shuttering can be removed: the concrete has become self-supporting. (Fig. 1.)

We have been able to make the full analysis of the nature of the physical phenomenon which allows of fresh concrete becoming self-supporting in a few moments at heights which in practice have exceeded 16 ft. We will summarise it:

As soon as the filter has been connected with the source of vacuum it communicates the fall of pressure of which it is the seat to the interstitial fluid of the fresh concrete in contact with it; gradually the "depression" reaches the whole system of minute channels of the concrete, which we may call the "circulatory system" of the concrete in the same way as the assembly of the aggregates is called the skeleton.

The penetration of this wave of depression follows the general laws of Fourier which govern the penetration of heat into solids; that is to say, the speed of penetration is first of all higher and slows up with time. In practice, the speed of the wave of depression varies from $\frac{3}{4}$ inch to $\frac{1}{2}$ inch a minute.

Fig. 1.—Immediately after the treatment, the shuttering is stripped: the concrete has become self-supporting. The pillars are part of a large water reservoir (Belgium)

What happens in a given zone of the fresh concrete when the depression reaches the interstitial fluid? We must go back to "Soil Mechanics" ("theory of consolidation" of Terzaghi) and note that in a given zone of fresh concrete the total pressure is the resultant of an intergranular pressure supported by the skeleton of the aggregates, and a fluid or interstitial pressure prevailing in the minute channels. This total or resultant pressure counterbalances the total load: the atmospheric pressure plus the self-weight of the concrete above the zone considered. In a zone not yet reached by the wave of depression, the distribution of the total pressure between

**Paper to be read before a Joint Meeting of the Institution of Structural Engineers and the British Section of the Societe des Ingenieurs Civils de France at 11, Upper Belgrave Street, London, S.W.1, on Thursday, November 27th, 1952, at 6 p.m.*

its two components is as follows: The interstitial pressure is equal to the atmospheric pressure plus the hydrostatic pressure due to the interstitial fluid located above the zone considered; its participation is therefore known. The intergranular pressure takes the rest of the load, which is generally low in relation to the interstitial pressure.

By putting the fluid under depression this equilibrium is disturbed and a profound change in the distribution of pressures is brought about. The interstitial fluid pressure is, in effect, reduced to a point of becoming almost nil and as the total load (atmospheric pressure and self-weight of the concrete) has not changed, the intergranular pressure increases considerably as the skeleton is bound to bear what the fluid no longer supports. It is thus subjected to an intense compacting effect and subsides in the endeavour to regain equilibrium. All the solid elements of the skeleton tend to approach nearer to each other as if there were an internal attractive force between them: the skeleton of the concrete "contracts" by a mechanism very similar to the shrinkage in course of crystallisation, with the difference that it has for its seat a concrete in the fresh state.

The contraction of the skeleton reduces the interstitial spaces and therefore reduces the volume left at the disposal of the water-cement paste. As this reduction of volume is imposed on a paste in contact with the filtering lining of the shuttering, the paste expels a part of its water through the filter and at the same time concentrates.

The concentration of the paste changes the structure of the latter. In effect, the paste before treatment is a "suspensoid," that is to say, a fluid comprising particles of cement in suspension; the particles are independent of each other, fall slowly according to the Stokes law (somewhat perturbed by electrostatic interaction) and would no longer remain in their respective positions if the water were suddenly to disappear.

By the phenomenon of concentration, the particles of cement are brought together to the point of forming a "micro-skeleton," that is, a structure similar to that which in the concrete, is formed by the aggregates; they are no longer independent of each other, no long "fall" according to the law of Stokes, and could remai in equilibrium if the water should disappear.

At the end of the treatment, the concrete has thus composite skeleton formed of both aggregates an cement particles. The interstitial fluid which in th untreated concrete is a water-cement paste, is her simply water.

At the moment the treatment is completed and th action of the vacuum is interrupted, on the surface the treated concrete at the outlet of the fine ducts the micro-skeleton (which are a few μ in diameter appear an infinity of menisca of very marked curvature these menisca are directly "hooked" to the peripher particles of cement and give rise to a general capillar tie. The material, under the effect of this tie, has suc cohesion as to have the character of a solid; it is in fac a "pseudo-solid" in the same way as clay. Th compression strength of fresh treated concrete—befor any crystallisation of cement—is of the order of 20 lb sq. in.; that is not strong, but it does allow the imm diate removal of the shuttering of elements 18 ft. high.

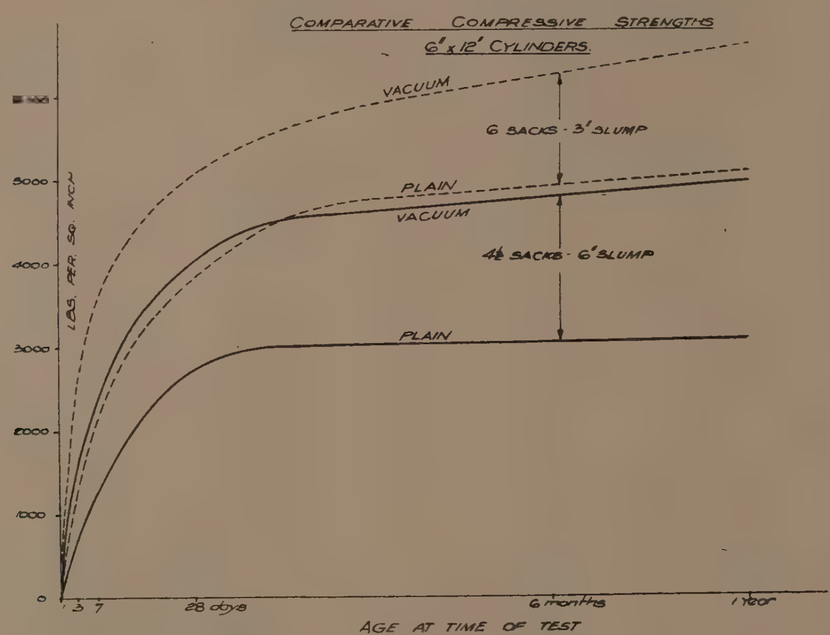


Fig. 2—(Tests made by E. L. Conwell & Co. for Vacuum Concrete Inc.)

This phenomenon of pseudo-solidification introduc into the life of the concrete, between the phase of th fresh, soft, untreated concrete and the solid phase crystallised concrete, a new phase which offers th builder many possibilities. The shutters, instead playing—as usually—a passive part of fairly lor duration (they prevent the concrete running before it crystallised), acquire an active part which they fulfil a very short time.

III.—Characteristics of the Treated Concrete aft Hardening

The concentration of the paste during treatment wi simultaneous expulsion of water leads to a mark reduction in the water/cement ratio. For example, has been possible to bring this down from 0.46 to 0.3 All the characteristics of the hardened concrete essentially functions of the value water/cement—a thus improved.

Fig. 2 gives some results of tests made in Ameri which show the comparative increase of compressi strength of a treated concrete and of the same concre

ntreated. In general, the strength is increased by oo per cent. in the course of the first seven days as ompared with the strength of untreated concrete : the 8 days strength is reached in eight to nine days.

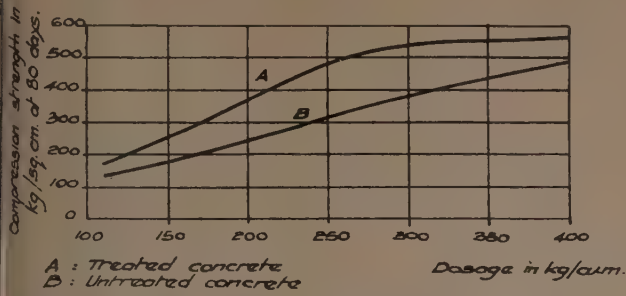


Fig. 3.—Tests made by the Building and Public Works Laboratory, Paris. Variation of strength of a 10 cm. slab made of concretes of identical plasticity, at various cement contents

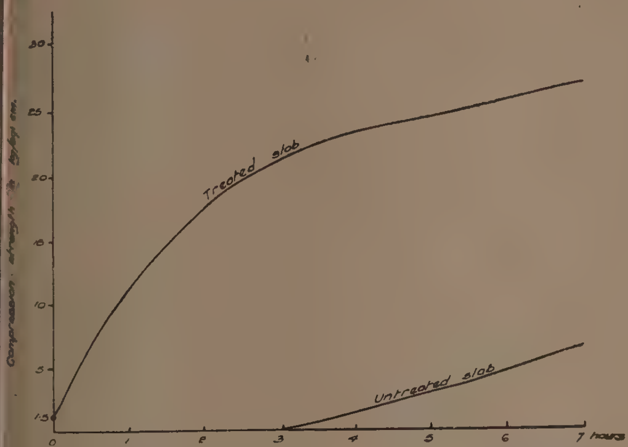


Fig. 4.—Hardening of a 10 cm. slab. Cement content 250 Kg/cu.metres. (Tests made by the Laboratory of Building and Public Works, Paris)

shows the strength at 90 days of concretes of differing cement contents. It is shown that with cement contents of less than 300 kg./cu. m. strengths are obtained equal to those of untreated concretes dosed at 400 kg./cu. m.

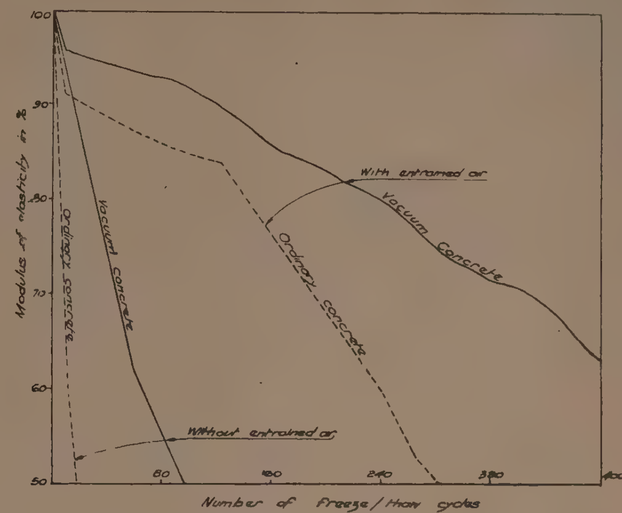


Fig. 5.—Variation of the modulus of elasticity under the action of frost. (Tests made by the U.S. Corps. of Engineers)

It is interesting to note that an appreciable strength appears in the course of the first hours as may be judged from Fig. 4.

The tensile strength is also increased ; recent tests made in Italy with richly dosed concretes (400 kg./cu. m.) —therefore of rather exceptional quality—give the following :—

Flexural strength :

840 to 980 lb./sq. in. in 7 days.

1540 to 1890 lb./sq. in. in 28 days.

The resistance to freezing and thawing is improved, but this improvement is less than that which results from

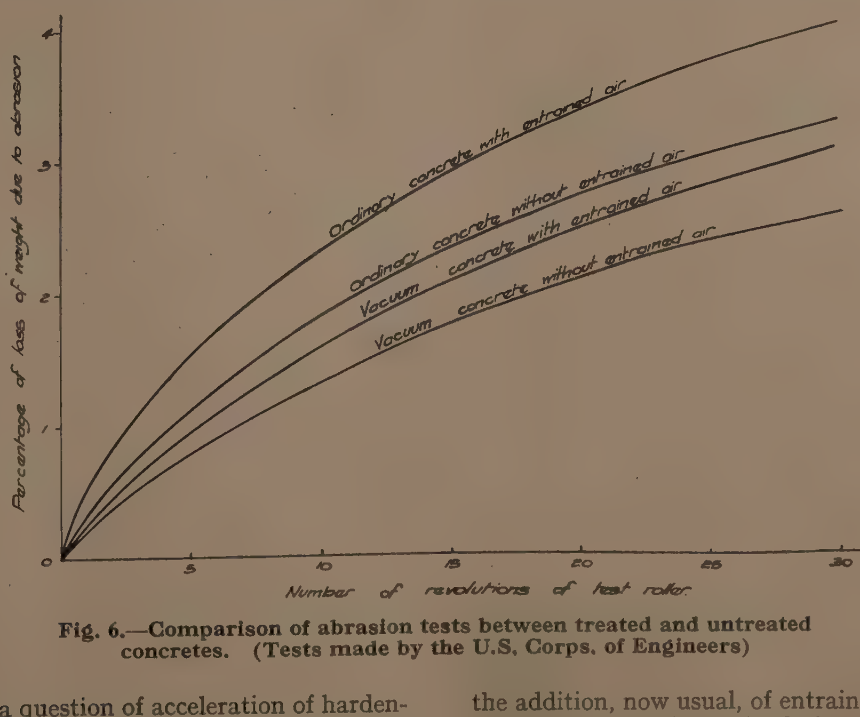


Fig. 6.—Comparison of abrasion tests between treated and untreated concretes. (Tests made by the U.S. Corps. of Engineers)

But it is not only a question of acceleration of hardening but also increase of the final strength : the latter is increased, usually from 1200 to 1500 lb. sq. in. We reproduce in Fig. 3 a curve taken from tests by the Public Works and Buildings Laboratory (Paris) which the addition, now usual, of entrained air in the concrete. The association of entrained air with the treatment has produced an absolutely remarkable improvement in frost resistance (see Fig. 5) ; this is important when good resistance to erosion is at the same time required.

In effect, entrained air reduces the latter and the treatment corrects this effect due to the increased resistance to erosion which it provides (it is this property which has led to the use of the vacuum treatment for numerous spillways (Fig. 6.)

Adherence of the treated concrete to reinforcement or to old concrete (repairs) is excellent.

Impermeability is increased : tests which have just been made in Paris on reinforced concrete pipes of 60 in. diameter and 5 in. wall thickness have shown that under prolonged water pressure of 5 atmospheres, no moisture appeared on the outside wall. In Italy pipes of 104 in. diameter, prestressed by means of a spirally wound binding are in use at 12 atms. inside pressure.

The modulus of elasticity of treated concrete is always better than that of untreated concrete, as is shown by Fig. 7.

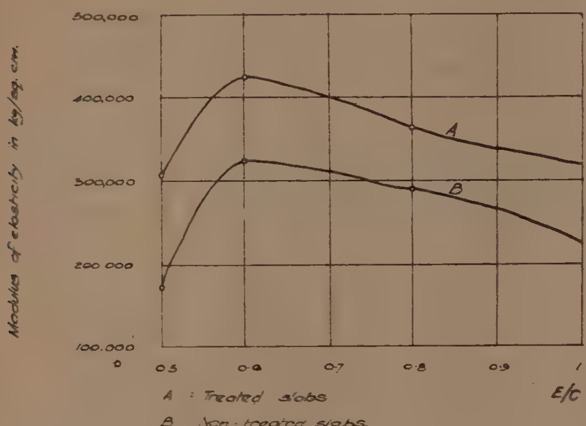


Fig. 7.—Variation of modulus of elasticity of a series of 10 cm. slabs. Cement content 300 Kg/cu.metres. Treated and not treated with variable water/cement ratio. (Tests made by the Building and Public Works Laboratory, Paris)

The surface of the treated concrete is free from voids, bubbles, or honeycombs. It generally happens in the course of pouring concrete that air bubbles are entrapped along the shuttering and even good vibration does not always eliminate them. In the case of the " active " (vacuum-lined) shuttering, the moment the treatment is commenced, the bubbles are expelled through the filter and the spaces left are entirely filled with paste.

Nevertheless, it must not be thought that the treated concrete is a deaerated concrete. In effect, the treatment does not result in a " suction " phenomenon but in a phenomenon of compaction in which the internal bubbles—held in place by the play of capillary forces—have no reason for expulsion (except in contact with the filter surfaces). Vibration remains the sole means of removing internal bubbles or in any case the larger of them ; thus, this is used in a very effective manner in conjunction with the vacuum treatment.

IV.—Advantages of the Vacuum Concrete Processes

These advantages result on the one hand from the application of the treatment to the fresh concrete and on the other hand from a set of devices also based on the use of atmospheric pressure.

(a) Various Methods of Applying the Treatment

The manner in which the treatment of the fresh concrete is applied in practice—that is to say the filters are introduced with a view to diffusing the depression in the minute channels of the concrete—varies accordin to the form of the elements to be treated.

In the particular case of elements or structures which at the moment when the concrete is poured, have a large, free upper surface (floors, flagging, big-spar arches, slabs, etc.), the treatment is applied by mean of extremely simple and light mats which are placed o the surface of the concrete as soon as it is poured—bearing in mind that it is not necessary to cover th whole surface at the same time. Starting up the vacuun causes the concrete to be compressed by the mat at pressure of almost a ton to the square foot. Th compacting in this case is associated with a small down ward movement of the mat of the order of 4 per cent. o the thickness of the treated concrete in ordinary cases

When it is a case, as is most general, of elements cas in a shuttering or in a mould (walls, pillars, pipes cas vertically, small span arches, prefabricated elements o various types, etc.), the treatment device is still simpler it suffices to cover the inner face of the mould or shutter ing with wire mesh and a fabric of gauze. In this cas the treatment device—that is to say, the lined mould—applies no compacting actions to the concrete (as i the case of mats placed on the surface) ; the mould thu comprises no moving parts and it is the direct actio of the atmospheric pressure on the free upper surfac of the concrete which causes the compaction ; th concrete itself " contracts " within the non-deformabl mould.

(b) Advantages Resulting from the Treatment

A certain number of advantages are found in th various applications : improvements of quality alread mentioned, but above all, and more important, genera economic advantages as follows :—

Saving of labour and power of vibration : due t pouring a plastic concrete (and finally obtaining—i this case not when the pouring is completed, but at th end of the treatment—the most compact concrete tha could be wished for, as it is absolutely no-slump).

Economy in cement : normally, except in specia cases, the cement content can be reduced by 25 per cent even though finally obtaining a stronger concrete.

Saving of time : by the more rapid readiness fo service, due to the accelerated hardening, both of pre fabricated units and of works poured *in situ*.

In the case—the most frequent—of elements cast i " active " shuttering, that is to say, treated by means c a simple lining of the shuttering, very great saving i shuttering is added to these general advantages : i effect, the shuttering is free at the end of the treatmen and can be used again immediately. The number c casings required is in some cases reduced by as muc as 80 per cent., whilst the additional unit price resultin from adapting the shuttering to make it " active " doe not exceed 20 per cent. to 25 per cent.

An economy in shuttering is also obtained in the cas of reinforced concrete floors or large span arches cas *in situ* and surface treated by means of mats. In thi case the saving of shuttering is not as big, as it does nc result from the phenomenon of pseudo-solidification—which would not allow in this case of immediate remova of the shuttering—but from accelerated hardening ; th shuttering can be removed two or three times soone thus affording a saving in shuttering of the order c 50 per cent. (in this case it should be emphasised tha the shuttering utilised differs in no way from the usua shuttering, as it does not exert the treatment).

In the case of floor slabs, terraces or floors which ar to have a " finished " surface (for example, when th floors are to be provided with a covering attache

ectly to the concrete) for which normally a mortar eed bed must be applied, the treatment completely minates the need for such an operation ; the concrete, mediately after treatment, whilst compact to the tent that it can be walked on without leaving an

Fig. 8.—Treatment of a terrace. Every few minutes one of the treatment mats is shifted. Note the workman on the left walking on the freshly treated concrete without leaving a trace (France)

nprint, is fresh, and immediate trowelling produces a urface to specification without the addition of cement r mortar. Instead of having an added coat or finishing, perfect monolithic paving is obtained. Vacuum-

Fig. 9.—Immediately after treatment, the treated concrete is trowelled

treated surfaces, moreover, give off no dust, are resistant to shock, watertight and are not subject to cracking (less shrinkage).

A final point, but not the least : in the field of moulded or prefabricated products, Vacuum Concrete means a saving in the areas required for production and stocking, due to the products being disposed of two or three times more quickly.

This rapid disposal results from the more rapid hardening of the concrete treated and also in many cases from the use of the Vacuum Lifter, a handling equipment forming parts of the same technique of the use of vacuum.

(c) Various Devices of Vacuum Concrete Technique

Various devices, other than those for the treatment of the fresh concrete, form part of the Vacuum Concrete technique :

" Vacuum Lifter " : handling by vacuum has been already utilised in various industrial fields—in particular for handling large glassplates. The vacuum lifter is an apparatus specially adapted to handling concrete units, provided in the case of units of large dimensions with a rigid frame which allows of handling without applying stresses to the concrete.

A group of suction heads of very simple type, judiciously distributed, render the stiffener frame solid with the parts to be handled, the inertia of the frame being shared by the concrete part. When lifting, it is the frame which takes the efforts so that the concrete is not subject to the usual concentrated stresses around the points of attachment or to the bending moments between these points. Due to this, the Vacuum Lifter, apart from the fact that it allows for removing concrete parts earlier from their moulds, allows for complete elimination of additional reinforcing iron for handling ; for pieces of prefabricated reinforced concrete of large dimensions, this often represents 25 per cent. of the steel.

The Vacuum Holder is used for fixing the shuttering to the ground or to other shuttering or for fixing vibrators to the shuttering.

Shuttering without external struts : by a special arrangement of the filter lining, the treatment is applied as and when the concreting progresses in small lifts and the hydrostatic thrust of the concrete is progressively suppressed in the strips subjected to the action of the vacuum ; simultaneously the shuttering is firmly applied to the concrete of these same strips by atmospheric pressure, which replaces and renders redundant external staying.

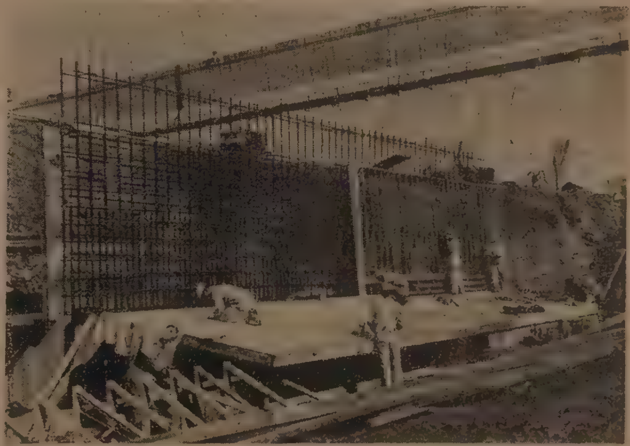


Fig. 10.—A retaining wall of the Friant Kern Canal (U.S.A.). Due to sub-division of the filtering device into horizontal strips which are put under vacuum successively, practically all the thrust of the concrete is eliminated (shuttering without stays). The workmen are fixing the filter cloth on the " active " shuttering

" Climbing Shuttering," applicable when high walls, silos, etc., are being poured. In this case, two strips of " active " shuttering are used of relatively small height (2 to 3 ft.) ; the strip " A " being in place, the concrete is poured to that height and the treatment commenced ; as soon as this strip is under vacuum, the strip " B " is fixed above it and the concrete for the second strip poured and its treatment commenced. In the meantime the concrete of the strip " A " has pseudo-solidified and the strip " A " is stripped to be fixed in turn above the strip " B " ; the work is continued in the same way.

Welding the Concrete (Joint Closure) : when it is desired to assemble two units of prefabricated wall, an " active " shuttering is fixed true with the joint (level or at an angle) held by holder to the wall units. The joint is filled and after treatment of a quarter of an hour, the

vacuum is disconnected both from the holder and from the "active" part of the shuttering and the shuttering is stripped. As the treated concrete of the joint will have very little shrinkage, the causes of parting between

Fig. 11.—Corner welding. Two prefabricated walls had been put in place. Between them was a joint. The fresh concrete poured into this joint being self-supporting after 15 minutes' treatment, the special shuttering was immediately removed and placed on the following corner. The workman is smoothing off the concrete of the joint which is fresh and soft. (U.S.A.)

it and the prefabricated units already hardened are eliminated ; hence no cracking.

V.—Fields of Application

Vacuum Concrete has three wide fields of application :

Products moulded in concrete, reinforced concrete or prestressed concrete ;

Public works, particularly hydraulic works ;

Building—above all when prefabricated.

Fig. 12.—Pipe 24 in. diameter and 6 ft. high stripped after eight minutes' treatment. (France)

In view of the considerable number of applications possible, only a few typical cases can be illustrated, as shown in Figs. 12-25.

As will be seen from these photographs, the processes are generally applied to elements :

Of relatively simple form, when lining the moulds with the filter is simple ;

Of not too great thickness—generally less than 1 ft.—so that the duration of treatment, which depends upon

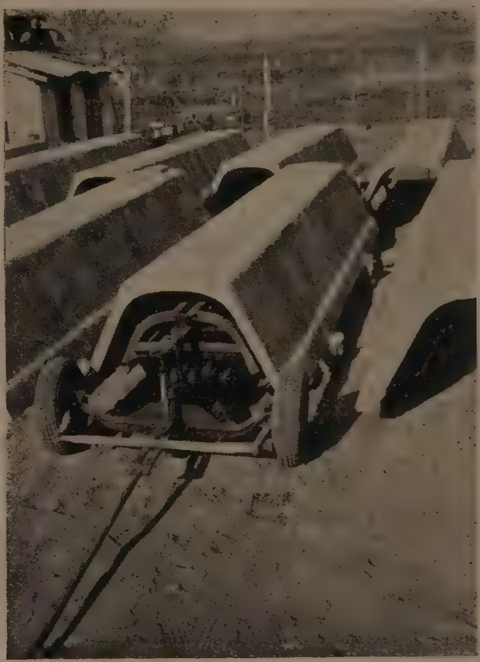


Fig. 13.—Prefabricated irrigation channels. These units are cast with their convacity downwards; the inner and outer shuttering being stripped immediately after treatment. By a special device, an inner face is obtained smooth as polished marble. Note the truck carrying one of the units. (Italy)

Fig. 14.—Multitubular slab for telephone cables. The treatment has been applied through the two larger faces. The cores being removed immediately, the holes remain truly round. (France)

the time of penetration of the depression into the concrete may not be too long ;

Of not too small size (such as building blocks) as for these vibration alone gives a very satisfactory production.

A certain repetition of the units is obviously favourable. Repetition, however, need not be absolute or

ery great since there is little special plant required, and n the other hand there is economy in usual equipment.

VI.—Equipment

As a general rule, the Vacuum Concrete processes are ased not so much on the introduction of the special quipment mentioned above, as on the saving of large art of the usual equipment (shuttering, moulds,

Fig. 15.—50 ft. electric standard. The portable mould is passed in front of the concrete mixer from which it is filled ; the treatment is applied by the shuttering of the vertical faces—including sockets—which allows an instantaneous stripping and shuttering. In 24 hours the standard is removed from its base. (France)

vibrators, etc.), and reduction in the installations (production and stockage areas), not to mention economies in cement and steel.

Lining of shuttering to render it "active" or the introduction of the treatment mats is of minor importance, the more so as the filter cloths can be repeatedly used (in some cases up to 200 times) ; replacement of the

Fig. 16.—Banks of the Ottmarsheim Canal (Alsace) covered with slabs prefabricated by the Vacuum Concrete processes. These slabs are taken out of their moulds after 16 hours. 5,000,000 sq. ft. of these slabs have been made and laid

cloths is compensated by the fact that there is no greasing of the active moulds.

The main new equipment introduced—essentially a specially equipped vacuum pump—generally represents an investment less than that of the usual compressor. This is why the "vacuum unit" is becoming, in the same way as the compressor, an everyday tool in the yard or on the site.

Fig. 26 shows the standard unit, normally used in works of medium size : power 9 h.p.

VII.—Some Questions Usually Asked

1. Is cement carried away during the treatment ?

Answer :

With the gauged filters used, no appreciable amount of cement is carried away—some 1/1000th of the cement

Fig. 17.—Articulated slabs (formed of slabs connected by rustless steel) for lining a canal. (France)

Fig. 18.—Banking of the Donzere-Mondragon Canal (Rhone), covered with articulated slabs. Note how they adapt themselves to the irregularities of the ground. 6,000,000 sq. ft. are in course of production

content, as there is no rapid flow of fluid in the mass but a slow percolation. The water removed comes out relatively clear ; it has a little fine turbidity of silt particles in suspension.

2. Does the evacuated water leave voids in the concrete?

Answer :

The water leaves no voids. The drying out must not be considered as the result of suction. Aspiration without compacting the solid particles could not remove water : thus a bottle full of water connected to a vacuum pump would remain full. If it were made of rubber, atmospheric pressure would compress it and allow the expulsion. With the vacuum treatment, the amount of water removed equals the contraction of the " skeleton " of the aggregate.

3. When the concrete under treatment has certain free surfaces, how is it that air is not drawn through them " through the concrete " ?

Answer :

This would happen if ordinary sand saturated with water were submitted to vacuum treatment : air would

Fig. 19.—Pipes 16 ft. high stripped after 27 minutes' treatment. A single mould produces five of these a day. These pipes are used under 12 atms. pressure. (Italy)

Fig. 20.—Installation producing the largest pipes in the world (50 tons each). Pipe dimensions : Diameter 14 ft., height 15 ft., wall thickness 1 ft. (Italy)

Fig. 21.—Drain covered by means of arch cast *in situ*. The " active " mould is of a length to correspond with one unit between joints (2.50m.). A single mould produces six units a day. (France)

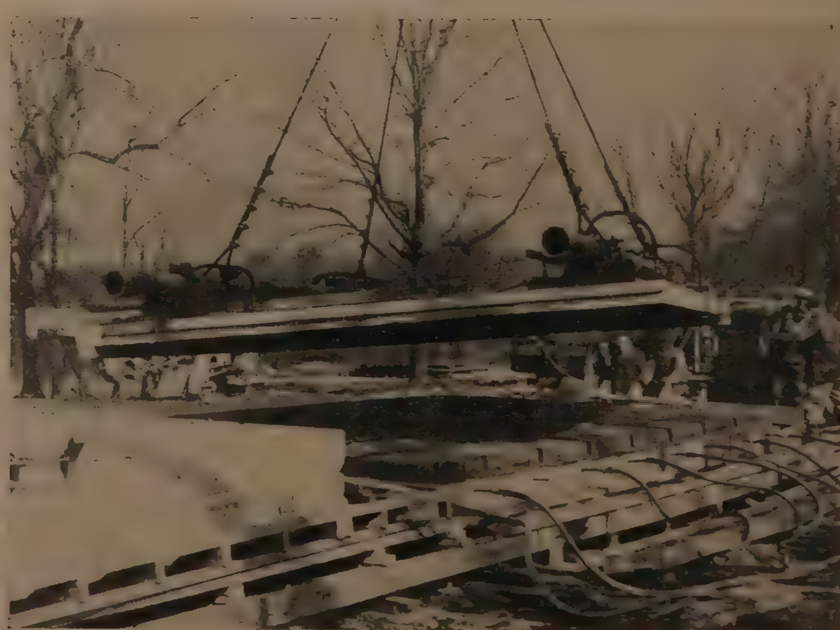


Fig. 22.—Stripping a prefabricated unit of ribbed floor; this unit cast on a concrete mould is stripped after 16 hours by a Vacuum Lifter. (U.S.A.)

e drawn in and would replace the interstitial water which it would drive out completely. In the case of the oncrete, fine menisca form on the surface at the outlets f the minute ducts in the paste and their fineness—and heir curvature (radius of about 1 micron)—is such that he atmospheric pressure is insufficient to release them rom their initial position. They "close" the ducts ightly.

4. Is there a risk of removing too much water by too rolonged treatment?

Answer :

There is no risk. When the "skeleton" has reached ts most compact arrangement, the expulsion of water eases automatically even if the vacuum is maintained. The residual water is that which fills in the interstices of the concrete thus compacted ; experience has shown hat in all cases more water is left than is required for he hydration of the cement. In practice, the water/ ement ratio has never been brought down by the reatment below 0.30.

5. Are inequalities of treatment found in the concrete according to the distance from the linings or mats applying the treatment?

Answer :

Differences at the end of the treatment are negligible ; moreover, capillary circulation after the treatment tends to render uniform the water content in the concrete as a whole.

6. In the case of treatment of floors, does the pressure of the mats apply stresses to the stays?

Answer :

The phenomenon is entirely internal ; the mats tend to draw nearer the inner shuttering, crushing the concrete

Fig. 23.—Element of prefabricated wall put in place by Vacuum Lifter. (Colombia)

Fig. 24.—Stripping a thin arched roof. These elements are cast one over the other every two hours, taking advantage of the strength of the treated concrete already sufficient at this age

slab between them, but neither the inner shuttering nor the stays is subjected to additional stress.

VIII.—Summary Bibliography

Works in English :

Vacuum Concrete. Lockhart, W. F. Proceedings of the American Concrete Institute. Vol. 34, pp. 305-19. January-February, 1938.

Vacuum Processed Reinforced Concrete Pipe. M. W. Loving. ROCK PRODUCTS, Vol. 51, No. 1, pp. 162-65, 172. January, 1948.

Reinforced Concrete Bridge Decks Precast by Vacuum Concrete. Loving. CONCRETE, Vol. 57, No. 8, pp. 10-11. August, 1949.

Vacuum Processes Applied to Precast Concrete House. K. P. Billner and Bert M. Thorud. JOURNAL OF AMERICAN CONCRETE INSTITUTE, Vol. 21, No. 2, pp. 121-8. October, 1949.

Fig. 25.—Hollow floor units. 704 flats have just been built with these elements at Heliopolis. (Egypt)

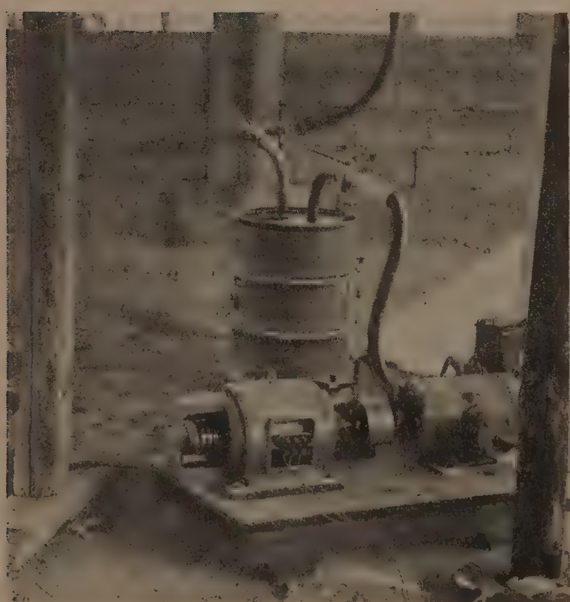


Fig. 26.—Standard vacuum unit. The 50-gallon barrel shows the scale. Right: the special vacuum pump ; left : a 9 h.p. electric motor

New Top Pressed on Concrete Road. Gunnar Johnson. ENGINEERING NEWS RECORD, Vol. 121, No. 17, pp. 531-2. October 27th, 1938.

Extracting Surplus Water from Concrete. Vacuum Concrete. Lockhart, W. F., Billner, K. P. CEMENT AND LIME MANUFACTURE, Vol. 12, No. 6, pp. 130-132. June, 1939.

Greater Strength, Hardness and Durability for Concrete. T. C. Creaghan, Building Science Abstracts V. XVIII (New Series), No. 8. August, 1945.

Vacuum Processing of Shasta Dam Spillway. C. S. Rippon. ENGINEERING NEWS RECORD, Vol. 134, No. 24, p. 829. June 14th, 1945.

Streamlined Vacuum Concrete Buntons for Mine Shafts. P. J. Doanides. JOURNAL OF AMERICAN CONCRETE INSTITUTE, Vol. 23, No. 4, pp. 309-319. December, 1951.

Applications of Vacuum Concrete. K. P. Billner. JOURNAL OF AMERICAN CONCRETE INSTITUTE, Vol. 23, No. 7, pp. 581-91. March, 1952.

Works in French :

Le Beton Sous Vide. I. Leviant and E. de la Sayette. LE GENIE CIVIL, Tome CXXV, No. 2-3-221, pp. 21-24. January, 1948.

Le Beton Sous Vide. G. Magnel. LA TECHNIQUE DES TRAVAUX, Vol. 24, No. 11-12, pp. 343-50. November-December, 1948.

Les Procedes Vacuum Concrete. I. Leviant and E. de la Sayette. LA TECHNIQUE MODERNE-CONSTRUCTION, Tome IV, No. 5, pp. 147-152. May, 1949.

Quelques Idees Sur Le Vacuum Concrete. I. Leviant and E. de la Sayette. REVUE DES MATERIAUX, No. 423, pp. 371-77. December, 1950.

Nouveaux Procedes de Traitement du Beton. R. L'Hermite. Annales de l'Institut Technique du Batiment et des Travaux Publics, No. 17. March-April, 1951.

Utilisation des Procedes Vacuum Concrete Dans L'Execution de Dallage au Port du Havre. I. Leviant and E. de la Sayette. LA TECHNIQUE MODERNE-CONSTRUCTION, Tome VI, No. 4, pp. 160-2. April, 1951.

Le Revetement du Canal D'Ottmarsheim en Dalles de Beton Prefabriquees Par Vacuum Concrete. Marc Bordeaux. LE GENIE CIVIL, Tome CXXVIII, No. 10, 3.301, pp. 181-86. May 15th, 1951.

L'Essorage du Beton Par le Vide et Quelques Applications du Vacuum Concrete. I. Leviant and M. Bordeaux BATIR, No. 15, pp. 8-13. September, 1951.

Works in German :

Zehn Jahre Talsperrenbau in den Vereinigten Staaten. J. D. Lewin. DIE WASSERWIRTSCHAFT—(sonderheft). 1949.

Works in Spanish :

El Vacio en la Construccion Como Elemento Nuevo de Trabajo. A. Semelas. REVISTA DE OBRAS PUBLICAS. May, 1950.

Works in Italian :

Tubazioni in Cemento Armato di Grande Diametro. F. Piccinini. COSTRUZIONI, pp. 19-34. March-April, 1952.

Prestressed Concrete Beams—A Rational Design Method

By Professor R. G. Robertson, M.A., M.I.C.E., M.I.Struct.E.

Part I (A) Simple Spans, Symmetrical Section

The present article describes the derivation of a method by which a direct answer for the beam dimensions may be found from one equation. Symbols used will be referred to when they occur and in a list on page 263.

The Moment of Resistance

In a composite beam of two materials, the external moment at any section (due to the loads and support reactions), produces an internal moment consisting of a single resultant force in each of the materials, one being tensile and the other compressive, of equal value.

The two forces produce a couple equal to the moment of resistance.

In a concrete beam the compressive force must lie within the core of the concrete section, if the stress is to be compressive throughout the section.

The tensile force in the steel is fixed in position, so that the lever arm may vary within the limits of the cable distance from the boundaries of the core of the section.

The force in the steel will vary from the initial force at the time of tensioning to the final force at time of full load, and intermediate stages need not be considered, as there will be smaller concrete stress for such cases. There will be a loss of steel force due to creep in the concrete, and a gain in steel force due to the application of load, and the difference between the two may amount to a loss of 10 per cent. to 15 per cent. according to the concrete used, the stresses, and the position of the cables.

The Two Critical Moments

(a) At the time of tensioning the only external moment will be that due to the weight of the beam,

$$M_1 = \rho A \frac{l^2}{8},$$

and for this case the maximum stress must occur at the bottom fibre : this stress must be the maximum allowable for the age of the concrete, C_1 ; any lower stress will result in an uneconomic section.

$$\therefore C_1 = P_1 \left(\frac{ey}{I} + \frac{1}{A}\right) - M_1 \frac{y}{I} =$$

$$\frac{P_1}{Ah}(e+h) - \frac{M_1}{Ah} \quad . \quad . \quad . \quad (1)$$

where P_1 = Initial tension in cables.
e = Distance from neutral axis to cables.
y = ,, ,, ,, ,, to bottom fibre.
h = ,, ,, ,, ,, to upper core point, $= \frac{I}{Ay}$

M_1 = External moment at time of tensioning.
A = Area of concrete section, omitting ducts.
I = Moment of inertia of concrete section, omitting ducts.
C_1 = Stress at bottom fibre at time of tensioning.

(b) At the time of maximum moment ($M_1 + M_2$), the lever arm should be at maximum which is found when the compression in the concrete lies at the upper core point.

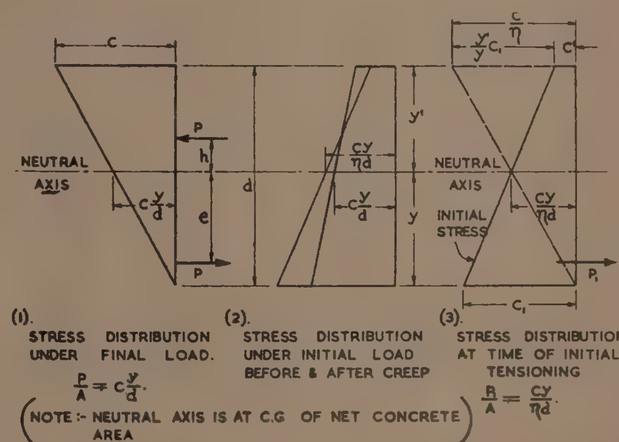


Fig. 1.—Stress Distribution

Let the stress at the top fibre be C and the force in the steel be $P = \eta . P_1$, where η is a factor allowing for the loss of tension due to creep. The stress at the neutral axis $= \frac{y}{d} C$

$$\therefore P = A \frac{y}{d} C \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

The " moment of resistance " is the steel force times its distance to the upper core point

$$M_1 + M_2 = P(e+h) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The Equation for the Beam Dimensions

Substituting for $(e + h)$ from (3) in (1)

$$\therefore C_1 = \frac{P_1 (M_1+M_2)}{P A h} - \frac{M_1}{Ah}$$

Substituting $\frac{P}{P_1} = \eta$

$$\therefore\ \eta A h C_1 = (1 - \eta).M_1 + M_2 \quad . \quad . \quad . \quad . \quad (3a)$$

Substituting $M_1 = \dfrac{A \rho l^2}{8}$

$$\therefore\ \eta A h C_1 - (1 - \eta)\, A\, \frac{\rho l^2}{8} = M_2 \quad . \quad . \quad . \quad . \quad (4)$$

Equation (4) gives the dimensions for a beam of any given shape of cross section for which A and h are functions of the depth. It is seen that the dimensions depend on the allowable initial stress C_1, the span l, and the superimposed load moment M_2.

The Minimum Stress in Top Fibre depends on the Maximum Stress

The compressive stress in the top fibre is a minimum at time of prestressing ;

From the geometry of Fig. (1)

$$C' = \frac{C}{\eta} - \frac{y'}{y} C_1$$

For C' to be compressive $C \geqq \eta \dfrac{y'}{y} C_1$. (5)

The Maximum Value of e

The value of e is given by equation (3)

$$e = \frac{M_1 + M_2}{P} - h$$

Substituting for P from equation (2)

$$e = \frac{M_1 + M_2}{A\,C}\ \frac{d}{y} - h \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Giving C its minimum value from equation (5)

$$\therefore\ e \leqq \frac{M_1 + M_2}{\eta A C_1}\ \frac{d}{y}\,{}_1 - h \quad . \quad . \quad . \quad . \quad . \quad (7)$$

The Maximum Stress in the Top Fibre

Economy of steel will result when e has its maximum value, and therefore C must be as small as possible, consistent with equation (5) and equation (6).

For economy of steel C should not exceed the value given by equation (5), or by substituting $e = e_0$, the value given by equation (6), whichever is larger governs.

$$C \geqq \left(\eta \frac{y'}{y} . C_1\right) \geqq \left(\frac{M_1 + M_2}{e_0 + h} . \frac{d}{Ay}\right) \quad . \quad (7a)$$

A greater value of C than the greater of these will involve a larger cable at less eccentricity than is economic.

$\dfrac{M_1}{M_2}$ Must be Large for Cable to have full Eccentricity

Let e have its maximum value e_0 in equation (7)

$$\therefore\ e_0 \leqq \frac{M_1 + M_2}{\eta A C_1}\ \frac{d}{y'} - h$$

From equation (3a)

$$A = \frac{(1 - \eta)\, M_1 + M_2}{\eta\, h\, C_1}$$

Substituting in above

$$e_0 \leqq \frac{M_1 + M_2}{(1 - \eta)\, M_1 + M_2}\ \frac{\eta\, h\, C_1\, d}{\eta\, C_1\, y'} - h$$

$$\frac{M_1 + M_2}{(1 - \eta)\, M_1 + M_2} \geqq \frac{e_0 + h}{h}\ \frac{y'}{d}$$

$$\frac{M_1}{M_2}\left\{1 - (1 - \eta)\frac{e_0 + h}{h}\ \frac{y'}{d}\right\} \geqq \left(\frac{e_0 + h}{h}\ \frac{y'}{d} - 1\right)$$

$$\frac{M_1}{M_2} \geqq \frac{\dfrac{e_0 + h}{h}\ \dfrac{y'}{d} - 1}{1 - (1 - \eta)\dfrac{e_0 + h}{h}\ \dfrac{y'}{d}}$$

$$\geqq \frac{\dfrac{e_0}{h} - \dfrac{y}{y}\,{}_1}{\dfrac{y}{y'} + \eta - (1 - \eta)\dfrac{e_0}{h}}$$

For a rectangular beam $\dfrac{M_1}{M_2} \geqq 1$ appr.

For a thin I beam $\dfrac{M_1}{M_2} \geqq \frac{1}{3}$ appr.

A smaller value of $\dfrac{M_1}{M_2}$ requires that the cable eccentricity is reduced.

Summarising the above

(a) when superimposed load is large compared to beam weight.

From (5) for no tensile stress in top fibre, minimum

$$C = \eta \frac{y'}{y} C_1$$

$$\text{Maximum } e \leqq \frac{d}{\eta\, y'\, C_1}\left(\frac{\rho l^2}{8} + \frac{M_2}{A}\right) - h \quad . \quad . \quad (7)$$

From equation (2), $P \gtreqless \eta A \dfrac{y'}{d} C_1$ . . . (7b)

(b) When superimposed load is small compared to beam weight.

Equation (7) gives too large a value for e ; e must be given its largest possible value e_0. There will always be compression in the top fibre. From Eq. (6) : substituting $e = e_0$ and $M_1 = \dfrac{A \rho l^2}{8}$;

$$\therefore C = \frac{d}{y} \frac{1}{e_0 + h}\left(\frac{\rho l^2}{8} + \frac{M_2}{A}\right) \quad . . . (8)$$

From Eq. (2) :

$$P = AC \frac{y}{d} \quad . . . . (8a)$$

These equations complete the beam design for which Eq. (4) gave the dimensions.

The Effect of Cable Ducts on the Section Properties

The above design equations (4), (7), (8) use expressions for A, h, y, y', which are functions of the depth of the beam for any given shape of beam, but it will be seen that the cable duct openings must be omitted from the section.

The ducts are grouted subsequently to tensioning the cables, and as the compressive stress in the concrete surrounding the ducts is reduced by the application of load, the grouting cannot take stress. The effect of the steel on the moment of inertia has been eliminated by using the known (approximately) final stress in the steel at the time of full load.

As the area of the ducts is small, the properties A_1, h_1, y_1, y'_1, of the gross section may be used instead of A, h, y, y', if suitable correction factors are introduced. Let the area of the ducts be qA_1.

Then $A = A_1 (1 - q)$.

The value of q may be 6 per cent. for Magnel type cables, but powers of q may be neglected.

The neutral axis of the gross section will be raised by qe by deducting the ducts so that

$$e = e_1 + qe$$
$$y = y_1 + qe$$
$$y' = y'_1 - qe$$

$$I = I_1 + A_1 q^2 e^2 - q A_1 (e + qe)^2$$
$$\doteq I_1 - q A_1 e^2$$

$$h = \frac{I}{Ay} = \frac{I_1 - q A_1 e^2}{A_1 (1 - q)(y_1 + qe)}$$

$$= \frac{y_1 h_1 - q e^2}{y_1 - q (y_1 - e)}$$

$$= h_1\left\{1 - q \frac{e^2}{h_1 y_1} + q\left(1 - \frac{e}{y_1}\right)\right\}$$

$$= h_1\left\{1 + q - q \frac{e}{y_1}\left(1 + \frac{e}{h_1}\right)\right\}$$

Thus h depends on e and equation (4) shows that the value of A must be increased if h is reduced.

The effect of the ducts in reducing h is greatest when e is greatest, and y_1 and h_1 are smallest.

h is least for a rectangular beam when $h_1 = \dfrac{d}{6}$

and $y_1 = \dfrac{d}{2}$

e has a maximum value of about $.8\, y_1 = .4d$.

For the worst case, therefore, for a rectangular beam, $h = h_1 (1 - 1.7q)$. The minimum duct size should always be used, but allowing for the largest ducts, with $q = 6$ per cent., then the least value of h is $h = .9h_1$. This value may be used in all cases as the error for smaller ducts at smaller eccentricity will give a beam section larger than absolutely necessary which will add to the factor of safety. Therefore let

$$A = .94 A_1$$
$$e = 1.06\, e_1$$
$$y = y_1 + .06 e_1$$
$$y' = y'_1 - .06\, e_1$$
$$h = .9 h_1$$

and the design equations (4), (7), (8), may be modified, so that the dimensions used refer to the gross section, and the ducts may be ignored.

Thus equation (4) becomes

$$.85\, \eta A_1 h_1 C_1 - .94\, (1 - \eta) A_1 \frac{\rho l^2}{8} = M_2 \quad . (10)$$

Equations (7), (7b) become :

$$1.06\, e_1 y'_1 - .06\, e_1^2 = - .9\, h_1 y'_1 + .054\, e_1 h_1$$
$$+ \frac{d}{\eta C_1}\left(\frac{\rho l^2}{8} + \frac{M_2}{.94 A_1}\right) \quad . . . . (11)$$

$$P = .94\, \eta A_1 \frac{y'_1 - .06\, e_1}{d} C_1 \quad . . . . . (12)$$

Equations (8), (8a) become :

(e_{01} = maximum possible value e_1)

$$C = \frac{d}{y_1 + .06\, e_{01}} \quad \frac{1}{1.06\, e_{01} + .9 h_1}\left(\frac{\rho l^2}{8} + \frac{M_2}{.94 A_1}\right) \quad . . . . (13)$$

$$P = .94\, A_1 C \frac{y_1 + .06\, e_{01}}{d} \quad . . . . (14)$$

Equations (13), (14) are only required when the value of e_1 given by equation (11) is too large for the size of the section, i.e., when $\dfrac{M_1}{M_2}$ exceeds the values given on page 260.

The Cable Profile

The maximum moment diagram for $\dfrac{M_1 + M_2}{P}$ will give the distance of the resultant force in the concrete above the cable at all points of the beam at time of full load, Fig. 2.

If a line is drawn at distance h_1 from the base, the intercepts will give the upper limit for the cable position relative to the neutral axis, so that tension cannot occur in the bottom fibre at time of full load.

The minimum moment diagram for $\frac{M_1}{P_1}$ will give the distance of the resultant force in the concrete above the cable at time of tensioning. Fig. 3.

If a line is drawn at e_1 from the maximum value of $\frac{M_1}{P_1}$ the intercepts will give the lowest limit of the cable position so that the bottom fibre stress shall not exceed C_1 at time of tensioning.

The diagram of Fig. 2 may be superimposed, as shown dotted, to give both limits of the cable profile.

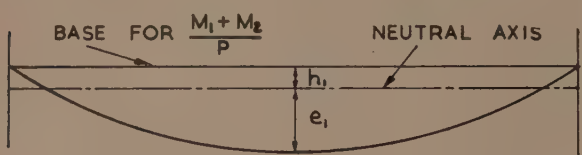


Fig. 2

EXAMPLE (1)

Rectangular Beam

See "Prestressed Concrete," Magnel, p. 28.
Span $70'$. $M_2 = 3 \times 10^6$ lb. in. per foot width.
$\eta = .85$, $C = 1500$*, $C = 1730$, $b = 12''$.

*It is necessary to discover if a smaller value of C may be beneficial.

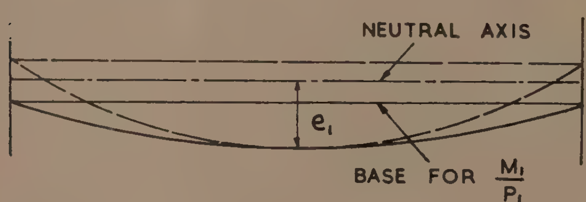


Fig. 3

$$A_1 = 12d,\ y_1 = y'_1 = \frac{d}{2},\ h = \frac{d}{6},\ \rho = \frac{1}{11.5}$$

Equation (10)

$$.85 \times .85 \times 12 \times 1730 \times \frac{d^2}{6}$$

$$- .94 \times .15 \times 12 \times \frac{840^2}{92} d = 3 \times 10^6$$

$$\therefore\ d^2 - 5.2\,d - 1200 = 0$$

$$\therefore\ d = 37.4''\ \text{(Magnel gives 38'')}$$

Use $d = 38''$, $h_1 = 6.3''$, $y_1 = y'_1 = 19''$, $A_1 = 456$

Equation (11)

$$.06\,e_1^2 - 20.1\,e_1 + \frac{38}{.85 \times 1730}$$

$$\left(\frac{840^2}{92} + \frac{3 \times 10^6}{.94 \times 456}\right) = 5.7 \times 19 - .33\,e_1$$

$$\therefore\ e_1^2 - 330\,e_1\,e_1 + 4{,}520 = 0\ \therefore\ e_1 = 14.5''$$

Magnel type cables require cover to centre $5''$ so e_1 must not exceed $14''$;

Hence equations (13), (14) must be used with $e_1 = 14''$

Equation (13)

$$C = \frac{38}{19.84}\ \frac{1}{14.84+5.67}\left(\frac{840^2}{92} + \frac{3 \times 10^6}{.94 \times 456}\right)$$
$$= 1390\ \text{lb./sq. in.}$$

*It is seen that a value $C = 1500$ is larger than necessary.

Equation (14)

$$P = \frac{.94 \times 456 \times 1390 \times 19.84}{38}$$

$= 310{,}000$ lb. (Magnel gives $.85 \times 372{,}000 = 316{,}000$).

The brevity of the calculation will be appreciated by comparison with that quoted.

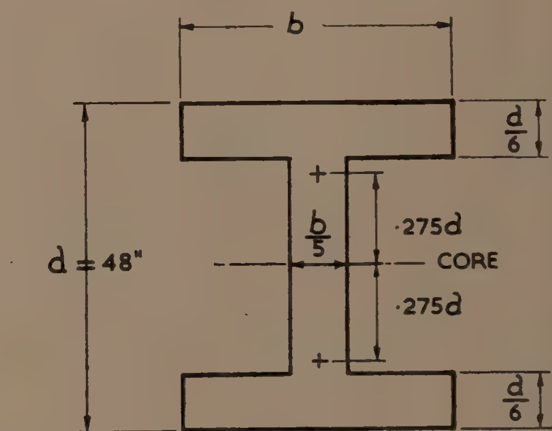


Fig. 4

EXAMPLE (2)

Symmetrical I Beam

See "Prestressed Concrete." Abeles. p. 41.
Span $= 80'$, $M_2 = 11.5 \times 10^6$ lb. in.
$\eta = .85$, $C_1 = 1500$, $d = 48''$.
Assume a section shape as Fig. (4).
$A_1 = .467\ bd = 22.4\ b$
$h_1 = .275\ d = 13.2$.

Equation (10).

$$.85 \times .85 \times 22.4 \times 13.2 \times 1500\ b$$

$$- .94 \times .15 \times 22.4 \times \frac{960^2}{92} b = 11.5 \times 10^6$$

$$\therefore\ b = 40''\ \text{web thickness} = 8''.$$

(Abeles gives $36''$, and $7\frac{1}{2}''$, but he includes the duct area in the stressed section, and if this is done the above equation would give $b = 34''$).

$$M_1 = 9 \times 10^6$$

$$\frac{M_1}{M_2} > \frac{1}{3}$$

$$\therefore \; e_1 > 20''.$$

Equation (13), (14) with $e_{01} = 20''$.

$$C = \frac{48}{25.2} \; \frac{23{,}600}{21.2 + 11.9} = 1{,}360 \text{ lb./sq. in.}$$

$$P = .94 \times 22.4 \times 40 \times 1{,}360\,(\tfrac{1}{2} + .025)$$
$$= 600{,}000 \text{ lb.}$$

(Abeles gives $.85 \times 690{,}000 = 590{,}000$ lb.)

It is observed that the results in the above examples are obtained without undue labour, or recourse to more than the 16 symbols required for the equations (10) to (14). These 16 symbols are recapitulated first.

16 Symbols for Final Design Equations

A_1 = Gross area of concrete section, including area of ducts.
b = Overall width of section.
C = Maximum top concrete stress at time of full load.
C_1 = Maximum bottom concrete stress at time of tensioning.
d = Overall depth of section.
e_1 = Cable distance below neutral axis of gross section area.
e_{01} = Maximum possible value of e_1 in regard to depth of beam.
h_1 = Upper core point distance from neutral axis of gross section area $\dfrac{I_1}{A_1 \; y_1}$
l = Span of beam in inches.
M_1 = Moment due to weight of beam.
M_2 = Moment due to superimposed load.
P = Tension in cables at full load.
P_1 = Tension in cables at time of tensioning.
y_1 = Bottom fibre distance from neutral axis of gross section area.
y'_1 = Top fibre distance from neutral axis of gross section area.
η = Ratio of cable tension at full load to cable tension at time of tensioning.
ρ = Density of beam in lb./cubic in.

11 Symbols also used but not Present in Final Design Equations

A = Net area of concrete section excluding ducts.
e = Cable distance below neutral axis of net section.
h = Upper core point distance, $= \dfrac{I}{Ay}$
y = Bottom fibre distance from neutral axis of net section.
y' = Top fibre distance from neutral axis of net section.
I = Moment of inertia of net section.
I_1 = Moment of inertia of gross section.
C' = Stress in top fibre at time of tensioning.
q = Ratio of area of ducts to gross area of concrete.
M = Maximum moment $= M_1 + M_2$
e_0 = Maximum possible value of e.

(B) The Best Shape of Section for a Specified Depth

It is seen from Equation (4), page 260, that the area of concrete section will be reduced if h, the upper core distance, is increased. Also on page 261, that a small addition in area, qA, at eccentricity e below the neutral axis will increase the value of h by

$$q\left\{\frac{e}{y}(e + h) - h\right\}$$

This shows that an increase in h can be obtained by adding area below the neutral axis, provided any addition is below the depth given by

$$\frac{e}{y} \gtreqless \frac{h}{e + h} \quad . \; . \; . \; . \; . \; . \; . \; . \quad (1)$$

In the Example (2), page 262 : $h = .275\ d$
$y = .5\ d$

An increase in h will be obtained by increasing the bottom flange width, which will result in a decrease in the area of concrete.

Example (3)

Modification of Example (2) with thinner flanges and bottom flange twice the width of the top flange. Fig. 5.

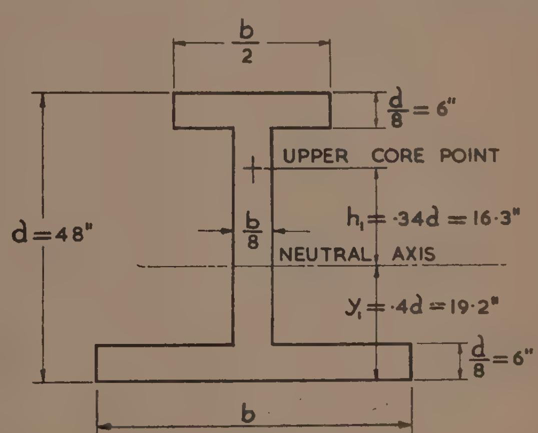


Fig. 5

See "Prestressed Concrete," Abeles, p. 43, Example B2.

$A_1 = .28\ bd, \quad y_1 = .4\ d = 19.2'', h_1 = .34\ d = 16.3''$

$l = 960'' : \quad M_2 = 11.5 \times 10^6$ lb. in. : $\eta = .85$

$C_1 = 1500$ lb./sq. in. $\rho = \dfrac{1}{11.5}$ lb./cu. in. : $d = 48''$, $e_{01} = 16.2''$

**C* depends on the properties of the section and cannot be assumed as 2250 lb./sq. in. (Abeles) but may be checked that it does not exceed this.

Equation (10), page 261.

$$b(.85 \times .85 \times .28 \times 48 \times .34 \times 48 \times 1500 - .94 \times .15 \times .28 \times 48 \times \frac{960^2}{92}) = 11.5 \times 10^6$$

$b = 53''$ (Abeles gives 48")
$A_1 = 710$ sq. in.
$A = 670$ sq. in. (Abeles gives 650 sq. in.)

Equation (13).

$$^*C = \frac{48}{20.2} \; \frac{1}{17.2 + 14.7}\left(\frac{960^2}{92} + \frac{11.5 \times 10^6}{.94 \times 710}\right)$$
$$= 2030 \text{ lb./sq. in.}$$

Equation (14).

$$P \; .94 \times 710 \times 2030 \; \frac{20.2}{48}$$
$$= 570{,}000 \text{ lb.} \quad \text{(Abeles gives } .85 \times 692{,}000 = 590{,}000\text{).}$$

If flange and web thickness had been assumed as $\frac{d}{6}$ and $\frac{b}{6}$ respectively, the width b would have been 46″ and the cable tension 630,000 lb. : this would have compared more exactly with Example (2) and shows a saving of concrete of 12 per cent. compared with the symmetrical section of the same depth and flange thickness.

The thinner flanges $\frac{d}{8}$ show a further saving in concrete

As the resultant concrete compression force may vary its eccentricity between the core points the greatest range in internal moment allowable is $\frac{CAH}{2}$; the range in external moment is due solely to superimposed load and may be found for any given loading and arrangements of spans, and its maximum value may be denoted M_6 ; then

$$\frac{CAH}{2} \geqq M_6. \qquad (2)$$

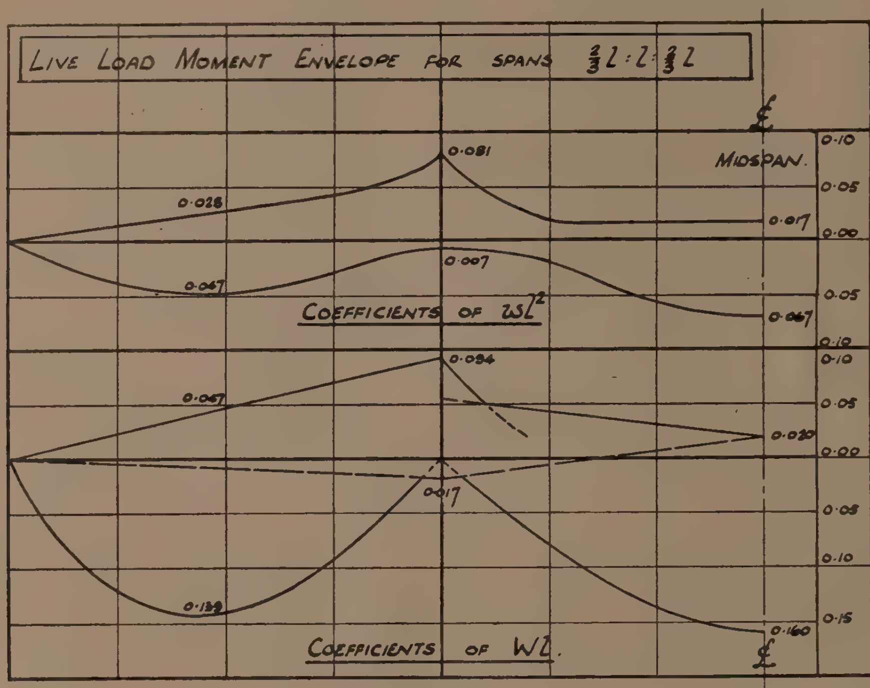


Fig. 6

area of 12 per cent. and a reduction in cable tension due to the reduced dead load and increased " core distance " lever arm.

It is suggested that the above method is quicker, having one equation for the beam dimensions, and using few symbols, and that it is an advance on any yet seen by the author.

Part II. Continuous Beams of Uniform Symmetrical Section

A. Cast In Situ Beams

(1) The external moment at any section equals the internal moment ; this is given by the cable pull times the lever arm, i.e., the cable distance from the resultant concrete compression.

As the cable pull is sensibly constant throughout the length of the beam the lever arm is proportional to the external moment, and is not a constant fraction of the depth of the steel as for normal reinforced concrete.

The lever arm may be found by dividing the external moment by the cable pull.

The stress distribution at the sections under maximum moment should range from O to C at the extreme fibres of the section, to give minimum steel area with no tensile stress, so that the cable pull is given by

$$P = \frac{CA}{2} \qquad (1)$$

As the maximum lever arm is $(e_0 + h)$ the greatest internal moment allowable is $\frac{CA}{2}(e_0 + h)$. The greatest external moment allowable is therefore greater than the range in moment allowable in the ratio $\frac{e_0 + h}{2h}$ which is 1.7 for a rectangular beam but is less for I sections. The greatest external moment consists of the sum of the moment due to superimposed load, and the moment due to the self weight of the beam, which latter may be found in terms of A, the area of section ; let M_4 represent the greatest external moment, then

$$\frac{CA}{2}(e_0 + h) \geqq M_4 \qquad (3)$$

Eq. (3) is not required unless

$$\frac{M_4}{M_6} > \frac{e_0 + h}{2h}$$

A beam of any given shape of cross section has values of A, H, e_0, h, given in terms of d the depth ; the dimensions may therefore be found from Eq. (2) or Eq. (3), whichever requires the larger section.

The critical sections both for range and for maximum moment are found either at a support or close to the centre of a span, and at the critical section the cable eccentricity is determined, as follows :

(2) The Cable Eccentricity

If range of moment governs the dimensions, then the prestress moment must equal (and must neutralise) the mean of the applied moments at the critical section, and this determines the cable eccentricity there. If maximum moment governs the dimensions, then the cable must have maximum allowable eccentricity at the critical section.

This means that for a uniform beam the moment of the area of the cable profile divided by the span length about the left end of the span for the left-hand span is equal in value but opposite in sign to the same expressions for the right-hand span, about its right-hand end ; each of these expressions represents the end rotation of the beam due to prestressing.

(b) *Two Spans*

For two equal spans, each of the expressions would have to be zero, and this is difficult if the cable is brought up to the top of the beam over the support; moreover, any support reactions created by prestressing reduce the

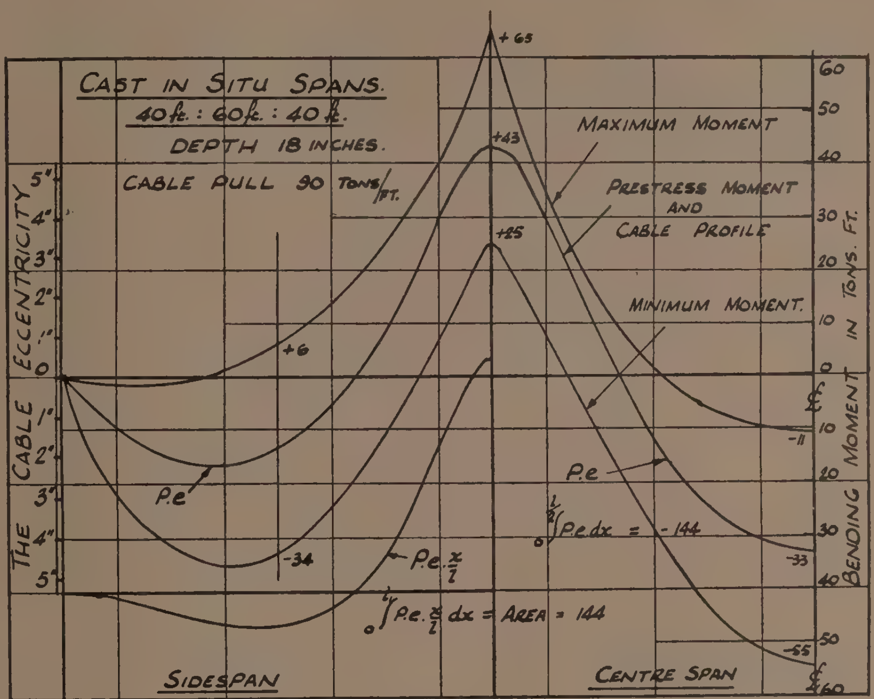


Fig. 7

At all other sections the cable may be placed at any eccentricity provided that the amount of the external moment, not neutralised by the cable pull, does not exceed that at the critical section ; this allows the cable profile to be adjusted, except at the critical sections, and this adjustment allows secondary moments to be eliminated.

The procedure may best be carried out on a diagram showing the maximum and minimum external moment envelopes, Fig. 7.

If the prestress moment is created without causing secondary moments, then the cable curve will have the profile given by dividing the prestress moment by the cable pull.

(3) Elimination of Secondary Moments

The moment caused by prestressing a continuous beam will not be the cable pull times its eccentricity, unless the cable profile is so arranged that no support reactions are created when tensioning the cables.

(a) *Balancing the Cable Profile*

There will be no support reactions created by prestress when the beam is freely supported and when

$$\int_0^{l_1} \frac{Pe\ x_1\ dx}{EI_1\ l_1} + \int_0^{l_2} \frac{Pe\ x_2\ dx}{EI_2\ \ l_2} = 0$$

prestress moment at the centre support, so it appears that two equal spans do not give an economic solution for small span prestressed beams of uniform section, for this reason as well as that the moment range equals that for simple spans. Freyssinet has suggested adjusting the support reactions by jacking (JOURNAL INST. C.E., Feb., 1950), so that he could eliminate this defect, but such a procedure would not perhaps be justified with small spans.

(c) *Three Spans*

With three spans, however, the prestress can be applied without creating support reactions, by allowing rotation at the supports, thus permitting a $+ve$ value for

$$\int_0^{l_1} \frac{Pe\ x_1\ dx}{I_1\ l_1}$$

in the side spans (taking e as $+ve$ above the axis), Fig. (7) and adjusting the cable profile so that the end rotations of the centre span are identical in value to those for the side spans. Let Pe be the proposed prestress moment value at any point. For a symmetrical three-span beam the balance is rapidly estimated by summing $\frac{x_1}{l_1} e$ at each

tenth point of the first span, using half the first and last ordinates to the sum of the nine interior ordinate multiplying by $\left(\frac{l_1}{10}\right)$ and adding the result to the area of the e diagram over the first half of the central span. Fig. 9.

The total is to be made zero by adjusting the profile of the cable curve in the quarter span region without altering its values at the critical sections.

*This procedure normally requires a positive result for the side spans and negative for the centre span, which means a downward beam curvature in the side spans and upward in the centre span : if the downward movement were resisted by the centring, the beam would tend to lift at the supports and precautions to prevent this were necessary or cracking could occur. Such precautions are obviated when applying the method to a beam precast in sections and erected in one length, or as three simple spans, as described in Part II B.

(d) *Magnel's Cable Profile*

It is evident that a predetermined expression for the cable profile such as suggested by Professor Magnel in his book "Prestressed Concrete," will not enable this to be done, and the cable profile of fixed shape will involve support reactions and secondary moments which alter the desired prestress moments. The desired prestress support moments can be achieved by trial, as suggested by Professor Magnel, but this alters the midspan prestress moments and the method is more troublesome than the adjustment of the cable as now proposed, so that Pe represents the prestress moment at every section.

(4) Effect of Cable Ducts

Allowance for cable ducts may be made as in Part (1) by assuming the worst position of the ducts and using correction factors for the corresponding dimensions of the gross section as given on page 260, except that the value of the "core point" distance H is not given on p. 261 ; as the lower core point distance is unchanged by the presence of the ducts, hence

$$H - 1.9\ h_1 = .95\ H_1$$

Substituting these correction factors (see page 261, Part I) and noting that $(e_{01} + h_1)$ is always less than $(e_0 + h)$ and may safely be substituted in Eq. (3)

Eq. (1) becomes $P = .47\ CA_1$ . . . . . . (4)

Eq. (2) ,, $.45.\ CA_1\ H_1 \geq M_6$ . . . . (5)

Eq. (3) ,, $\left(\frac{CA_1}{2}\right)\ (e_{01} + h_1) \geq M_4$ . . . (6)

These equations define the section in terms of the gross area, and allow for the largest ducts at the greatest possible eccentricity.

(5) Economic Span Ratios for Three Spans

For small spans with comparatively large live loading, the range in moment will govern the size of beam required, see Para. (1), Part (2).

In comparing a three-span continuous beam with three simple spans, two cases arise

(a) when the total length to be spanned is given,

(b) when the central span length is given but the side spans may be any length to give the most economic solution.

(a) The economic ratio for the spans for a single point moving load is 1 : 1.2 : 1, which shows a 5 per cent. reduction in moment range compared to three equal simple spans of the same total length.

With uniform moving live load, however, the continuous beam gives 8 per cent. greater moment range than the three equal simple spans.

The economy of the continuous beam is therefore very questionable in case (a) except for large dead loads.

(b) When the length for the side spans is 2/3 of the central span, the moment range in the centre span is about 30 per cent. less than that for a simple span of the same length as the centre span, either for a uniform or for a point live load.

When the length of the side spans is 1/3 of the length of the central span, the moment range is about 40 per cent. less than for a simple span of the same length as the central span, either for a point load or a uniform live load.

Shorter side spans reduce the moment range for point loads still further but for a ratio less than 1/4, the range for uniform live load again increases and with short side-spans the ends of the beam would need anchorage against uplift.

It appears therefore that a continuous beam is more economical than simple spans if the centre span is considerably larger than the side spans. Example (4) illustrates this.

It will be seen from Appendix (2) that the support moment range is less than the central moment range for a single point load, and the support moment range is only about 5 per cent. larger than the central moment range for a uniform live load, for side-span length greater than 2/3 of the centre span length, so that for these spans a uniform beam will be more economical than a haunched beam, when live load governs the beam dimensions. Example (6) illustrates this.

(6) The Max. and Min. Moment Envelope for Moving Loads

The few published examples of moment envelopes found by the author are not applicable to the above cases.

The cable profile design requires the curve of the moment envelope, as well as the maximum and minimum moments at support and midspan.

The moment envelope for a movable uniform live load, and for a moving single point load is given in Fig. 6 for a side span 2/3 of the centre span.

(7) EXAMPLE (4) Fig. 7

HIGHWAY BRIDGE DESIGN FOR CAST IN SITU SLAB DECK

Spans 40′ ; 60′ ; 40′ working stress 2000 lb./sq. in.
Superimposed Load ; 20 lb./sq. ft. for surfacing, plus Live Load
220 lb./sq. ft., plus 2700 lb./ft. width point load.
The greatest range of moment occurs at midspan and

$$M_6 = 44 \text{ ft. tons.}$$

$$\text{Eq. (5) ; } .45 \times 2000 \times 12\ d \times \frac{d}{3} = 44 \times 2240 \times 12$$

$$d = 18''$$

NOTE : Part (I) gave 25 in. depth for a simply supported central span and 16 in. for the side spans, so that the continuous beam gives 15 per

cent. saving in concrete, and 25 per cent. saving in central beam depth.

The Maximum Moment occurs at a support :
Dead Load Moment : See App. (2) :

$$.074 \times 225 \times 60^2 \times \frac{1}{2240} = 27 \text{ ft. tons.}$$

Superimposed Load Moment plus Live Load Moment :

$$.074 \times 20 \times 60^2 \times \frac{1}{2240} +$$

$$(.081 \times 220 \times 60^2 + .094 \times 2700 \times 60) \frac{1}{2240} = 38 \text{ ft. tons.}$$

$\therefore$ Maximum Moment $M_4 = 65$ ft. tons.

This is less than 1.7 times M_6 so the above section is sufficient, see Para. (1), Part (2A).

$$\text{Eq. (4) gives } P = .47 \times 12 \times 18 \times 2000 \times \frac{1}{2240} = 90 \text{ tons.}$$

Dead Load Moment at midspan = — 18.
Maximum superimposed Load Moment = — 37
Minimum ,, ,, ,, = + 7

The prestress moment at the centre of the central span must neutralise the mean moment there, which is seen to be — 33 ft. tons, and the cable eccentricity must be

$$\frac{35}{90} \times 12 = 4.4''$$

The amount of applied moment not neutralised by prestress is $\pm$ 22 ft. tons.

At the support, the maximum moment is 65 ft. tons so that prestress moment must be (65 — 22) = 43 ft. tons and the cable eccentricity must be $\frac{43}{90} \times 12 = 5.7''$.

This is the largest eccentricity, and is .3 d only.

Under dead load alone, and with 15 per cent. increased cable pull at time of initial tensioning the support moment is

$$(27 - 43 \times 1.15) = -22 \text{ ft. tons.}$$

The initial cable pull is $90 \times 1.15 = 103$ tons.

The initial stresses are, therefore (allowing for cable ducts),

$$\pm \frac{M}{Ah} + \frac{P_1}{A}$$

$$\pm \frac{22 \times 12 \times 2240}{.85 \times 216 \times 3} + \frac{105 \times 2240}{.94 \times 216} =$$

$$+ 2210 \text{ lb./sq. in. or } + 70 \text{ lb. sq. in.}$$

Tension does not occur at any section and the initial stress is only 10 per cent. greater than the working stress in compression.

The cable pull of 90 tons requires the same amount of steel as for three simple spans but there is a saving in end anchorages, in concrete, and a 25 per cent. saving in central beam depth.

The cable profile is adjusted with due regard to the 22 ft. tons moment difference allowed between the prestress moment and the maximum or minimum moment at each section, as shown in Fig. (7).

(8) Effect of Increased Initial Cable Tension on the Initial Stresses

(a) *Initial Tensile Stress*

Under final loads no tensile stress has been allowed in the concrete.

Concrete tensile stress of $\frac{C}{10}$ has been suggested by Magnel as permissible, together with a reduction of 15 per cent. in the initial cable tension due to creep. See his book " Prestressed Concrete."

Let the initial cable pull be increased, at time of initial tensioning, by $.15P$, to allow for the difference between the loss due to creep and the gain due to load.

At a section where the cable has maximum eccentricity, and where the dead load stress might be zero at the extreme fibre opposite the cable the increase in initial pull of $.15P$ would cause initial tension of

$$.15 \frac{P}{A} \left(\frac{e}{h'} - 1 \right)$$

The maximum value of $\frac{e}{h'}$ is 2.5 which occurs in a rectangular section with maximum cable eccentricity.

Substituting this value :

$$\text{Initial tension} = .15 \frac{C}{2} (2.5 - 1)$$

$$= .11\, C$$

It appears therefore that provided no tension is allowed under working conditions, the increased initial pull will not cause undue tensile stress in the concrete ; moreover any cracks formed would close up as soon as the initial excessive cable pull was reduced by creep. No tensile stress may actually be involved as the cable eccentricity may not be its maximum, and the dead load stress may not be zero.

(b) *Initial Compression Stresses*

The maximum initial compression stress occurs at the section where the dead load compression stress is C.

Due to an increase in cable pull of $.15P$ there will be an increase in compression stress

$$.15 \frac{P}{A} \left(\frac{e}{h} + 1 \right) = .15 \frac{C}{2} \left(\frac{1.06\, e_1}{.9\, h_1} + 1 \right)$$

which has a maximum value $\frac{C}{3.4}$ when $\frac{e_1}{h_1} = 2.4$,

i.e., for rectangular sections with maximum eccentricity. Magnel indicates that the initial stress may be larger than the working stress by a similar amount, no danger being involved by overstress at this stage.

NOTE 1 : In both the above cases the additional stresses depend on the maximum value of $\frac{e}{h}$, which is often not as large as given above and is smaller for I

section beams, so that the initial stresses given above would be correspondingly reduced in most cases, see Example (4).

NOTE 2 : The same considerations apply to Part (I) in which some economy of section may result by allowing tensile stresses at time of initial tensioning ; the same dimensions will be found, however, when l/d is large, as no tension can occur in that case.

Design equations (10) and (11) of Part I may be used with $\eta = 1$, if C_1 is reduced suitably (20 per cent.).

*NOTE 3 : The considerations of Note 2 may be reversed and Equation (1) of Part I applied to a continuous beam to check that the initial stress C_1 is not excessive.

This could occur when the superimposed load consists largely of uniform dead load in which case the minimum moment will be the beam weight moment, which is the moment at the time of initial tensioning.

In such cases Eq. (1) of Part I should be used to determine the section, substituting the dead load coefficient of Appendix II for the fraction $\frac{1}{8}$ in the expression for the beam weight moment $A\rho\dfrac{l^2}{8}$ of Eq. (4) and (10) Part I, and using the maximum moment due to superimposed load for the continuous beam for M_2.

NOTE 4 : *The Range in Moment*

The range in moment due to live load and superimposed dead load will only be increased by the latter if the sum of the moments due to both, cannot change sign, in which case the range in moment equals the maximum moment due to both.

Part IIB. Continuous Beams Precast in Sections

(1) ERECTION USING TEMPORARY CABLES

*If temporary cables were used for the preliminary prestressing and removed when the permanent cables were installed, the equations of Part IIA would apply to the design of the beam and of the permanent cables. Temporary cables could be used to permit rolling in or other means of erection of the complete precast beam or for precasting the beam in several sections and its erection as one continuous span, or as three separate spans.

(2) ERECTION AS PREFABRICATED SIMPLY SUPPORTED BEAMS

If the preliminary cables were to remain permanently and erection as three simple spans was contemplated the design had to be modified as follows :

(a) *The Continuous Cable*

As there was no initial moment at a joint and as there must be no tensile stress at a joint, any continuous cable must be placed within the core of the section there ; hence the moment at a joint cannot exceed the cable pull times the core distance, and both the maximum external moment, and the maximum external range of moment M_5 must be less than the allowable internal moment.

$$\therefore\ M_5 < P_2 H \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

(b) *The Preliminary Prestress Cables*

The preliminary prestressing required for handling would create compressive stress in the spans, which must be added to that created at the joints by the continuous cables, so that a greater compressive stress would occur close to the joint than across the joint itself.

The preliminary compression stress should therefore be as small as possible and should be uniformly distributed at a section close to the joint, where the preliminary prestress force should therefore have zero eccentricity.

The minimum preliminary prestress required is given by the moment due to the weight of the beam during handling and erection as a simple span ; this is given by

$$P_0 = \frac{A\rho l^2}{8(e_0 + h)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

As the resultant compression force is at the upper core point, the residual moment is $-P_0h$, and is sagging.

(3) *If Stress at Joint Governs the Dimensions*

The maximum stress at the joint must not exceed $\left(C - \dfrac{P_0}{A}\right)$; this stress occurs when the resultant compression at the joint is at either core point so that the stress is $\dfrac{2P2}{A}$

$$\therefore\ \left(C - \frac{P_0}{A}\right) = \frac{2P_2}{A}$$

Substituting for P_2 from Eq. (7)

$$\therefore\ C - \frac{P_0}{A} > \frac{2M_5}{H}$$

$$\therefore\ AHC - HP_0 > 2M_5 \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

This is an equation for the dimensions (after substituting for P_0 from Eq. (8)).

(4) If Stress at Midspan Governs the Dimensions

The total cable pull at midspan will be $(P_0 + P_2)$ so that the allowable range in internal moment is $(P_0 + P_2)H$.

$$\therefore\ (P_0 + P_2)\,H > M_3 \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

If the resultant compression in the concrete may be at either core point then the maximum stress is given by

$$\frac{2(P_0 + P_2)}{A} = C \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

Substituting for $(P_0 + P_2)$ from Eq. (10) in (11)

$$\therefore\ AHC > 2M_3 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

There is a minimum limit for P_2 given by Eq. (7), so that the stress at midspan may never become zero, as envisaged by Eq. (11).

At midspan the prestress moment may always be arranged to neutralise the dead load and the mean superimposed load moments so that the remaining moment borne by the concrete does not exceed $\dfrac{M_3}{2}$

The cable pull at midspan is $(P_0 + P_2)$ so that the maximum stress is given by

$$\frac{P_0 + P_2}{A} + \frac{M_3}{AH} < C$$

ubstituting for P_2 from Eq. (7)

$$\therefore \quad \frac{P_0}{A} + \frac{M_5}{AH} + \frac{M_3}{AH} < C$$

$$\therefore \quad AHC - P_0H > M_3 + M_5 \quad . \quad . \quad . \quad . \quad (13)$$

This equation (13) applies when $M_3 > M_5$ but Eq. (9) pplies when $M_5 > M_3$. Eq. (12) will supersede both nly when

$$M_3 > M_5 + H\,P_0$$

ut this only applies to very small spans.

(5) Modifications to Allow for Ducts

The correction figures given in Part I, page 260, for onversion of the dimensions of the gross area to those of the net area, thus allowing for the ducts, will be maller for precast continuous beams as the cable ccentricity is less.

The value of e will not generally exceed h in this case, o the factor for h becomes: (see Part I, page 260, and Part 2, page 265).

$$h = .96\ h_1 = .48\ H_1$$
$$h' = h_1 = .50\ H_1$$
$$H = .98\ H_1$$
$$e_0 = 1.06\ e_{01}$$

The remaining factors are unaltered.

Substituting the dimensions of the gross area in the above equations

$$\text{Eq. (8) becomes } P_0 = \frac{.94\ A_1\ \rho\ l^2}{8(1.06\ e_{01} + .48\ H_1)} \quad . \quad (14)$$

For $M_5 > M_3$

$$\text{Eq. (9) becomes } .92\ A_1\,H_1\,C - .98\ H_1\,P_0 = 2\ M_5\ . \quad (15)$$

$$\text{Eq. (7) becomes } P_2 = \frac{M_5}{.98\ H_1} \quad . \quad . \quad . \quad . \quad . \quad (16)$$

For $M_3 > M_5$

$$\text{Eq. (12) becomes } .92\ A_1\ H_1\ C = 2\ M_3 \quad . \quad . \quad . \quad (17)$$

$$\text{Eq. (13) becomes } .92\ A_1\,H_1\,C - .98\ H_1\,P_0 = M_3 + M_5 \quad . \quad . \quad . \quad . \quad (18)$$

Eq. (15) or (18) must be compared with Eq. (17), whichever gives the larger dimensions must be used

$$\text{Eq. (10) becomes } P_2 = \frac{M_3}{.98\ H_1} - P_0 \quad . \quad . \quad . \quad (19)$$

Eq. (19), must be compared with Eq. (16), whichever requires the greater cable pull must be used.

(6) The Cable Profile

(a) If Eq. (16) governs the value of P_2 then the continuous cable must have an eccentricity at the supports such that the prestress moment there will neutralise the smaller of the following moments due to superimposed loading.

(1) Half the maximum moment.
(2) The mean of the maximum and the minimum moments.

Dead Load Moment will be eliminated as the joint will be closed while the dead load is supported by the beams as simple spans.

(b) If Eq. (19) governs the value of P_2 then the continuous cable must have an eccentricity at midspan such that the prestress moment there will neutralise the sum of—

(1) The mean of the maximum and minimum moments due to superimposed load.
(2) The residual moment—P_0h from beam weight as a simple span not already neutralised by the preliminary prestressing.

(c) The above fixes the prestress moment at supports when (a) governs or at midspan when (b) governs ; at all other sections the cable eccentricity may be varied between such limits that the amount of moment not neutralised by prestress does not exceed that at the critical section, as given by (a) or (b).

The procedure is best carried out on a diagram showing the maximum and minimum moments at every section due to the sum of—

(1) The moments due to superimposed load on the continuous beam,
(2) The residual moments due to dead weight on the simple spans after preliminary (handling) prestress, see page 268; these are $-P_0h$ at midspan, zero at supports, and a value in the sidespans that may be varied by varying the preliminary cable pull there, up to a maximum value P_0.

If the sidespans are less than 5/6 of the centre span in length, their residual moment due to dead weight may be made $+ve$ (if advantageous), see Fig. 8.

(d) The prestress moment is adjusted as allowed by (c) above so that no secondary moments are created by prestressing, as described on page 264; the added freedom of adjustment described in (C2) above makes several profiles available.

Example 5. Fig. 8.

Continuous Bridge Deck 40′ ; 60′ ; 40′ erected as simple spans of rectangular section 12 in. wide in contact laterally.

Superimposed Load ; 20 lb./sq. ft. surfacing, plus 220 lb./sq. ft. uniform live load and 2700 lb./ft. width point load.

$$C = 2000\ \text{lb./sq. in.} : A_1 = 12\,d : H_1 = \frac{d}{3} : h_1 = \frac{d}{6}$$

From Appendix (2) :

$M_3 = 44$ ft. tons $\quad M_5 = 40$ ft. tons.
Eq. (14) gives $P_0 = 56$ tons (P_0 is independent of the depth).
Eq. (17) gives $d = 18''$
Eq. (18) gives $d = 20.5''$
Take $d = 21''$; $\therefore H_1 = 7''$; $h_1 = 3.5''$
This is an increase of 3″ (which is 16 per cent.) more than the similar cast *in situ* span of Example (4), but a decrease of 4″ (which is 16 per cent.) less than the simple span.
Eq. (16) gives $P_2 = 70$ tons.
Eq. (19) gives $P_2 = 22$ tons.
$\therefore P_2 = 70$ tons.
Maximum Superimposed Load Moment at midspan = + 6 ft. tons
Minimum Superimposed Load Moment at midspan = — 38 ft. tons.
Mean Superimposed Load Moment at midspan = — 16 ft. tons
Residual beam weight moment at midspan
$.96\ P_0h_1 = -15$ ft. tons.

$\therefore$ Prestress moment at midspan = 31 ft. tons.
$\therefore$ Continuous cable eccentricity at midspan = —5.4″

The moment envelope shown in Fig. (8) allows for the same value of P_0 in the sidespans as in the centre span, so that the residual moment is +16 ft. tons. A suitable prestress moment curve for the continuous cable is also shown, which will create no secondary moments, and at the same time no greater moment remains after prestress than the 22 ft. tons which occurs at midspan.

The continuous cable requires +3″ eccentricity at the inner supports, which should be sufficient to separate it from the preliminary (handling) prestress cables which are at the centre line there ; it is also given $+2\frac{1}{4}''$ eccentricity at the ends where the preliminary (handling) prestress cables may be given — 3″ eccentricity so that the end anchorages will be sufficiently far apart : both

$\therefore A = 333$ sq. in. $\therefore H_1 = e_{01} = 12.4''$ $\therefore h = 6''$
Equation (16) gives $P_2 = 370{,}000$ lb.

The maximum eccentricity occurs at the supports where it is $\dfrac{M_5}{2\ P_2} = 6''$, i.e., at the upper core point.

The moment envelope is given in Fig. (9) allowing zero residual moment due to beam weight in the sidespans ; the same preliminary (handling) cable tension there as in the central span makes this possible due to the shorter length of the sidespans.

If the continuous cable is given 15 per cent. additional pull at time of tensioning, the compression stress at the support will be increased in the same proportion and will become 2870 lb./sq. in.

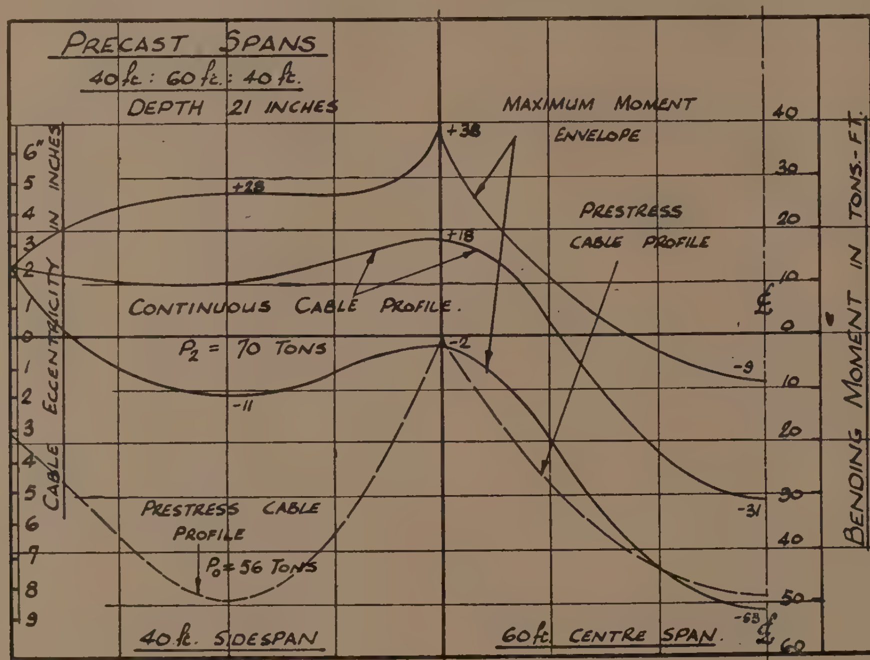


Fig. 8

these dimensions could be increased to $3\frac{1}{2}''$ if desired, without causing tensile stress or excessive compression stress, and alternative cable curves could be achieved by reducing P_0 in the sidespans, as long as the equality of

$$+\int_0^{l_1} e\,\frac{x}{l_1}\,dx \text{ and } -\int_0^{l_2} e\,dx \text{ in the centre span was}$$

maintained : the cable profile as shown had the minimum curvature of all possible variations and was preferable in this respect.

EXAMPLE (6) FIG. (9)

Continuous Beam of Spans 30′ : 36′ : 30′ erected as simple spans and subject to uniform superimposed dead load of 1050 lb./ft. and uniform live load of 2100 lb./ft.

$C = 2500$ lb./sq. in.

An I section similar to Fig. (9), is specified, and a width of 14″.

$A_1 = 11.4\,d\ ;\ e_{01} = H_1 = .4\,d\ ;$

From Appendix 2 :

$M_5 = 4.5 \times 10^6$ inch lb. $M_3 = 3.3 \times 10^6$ inch lb.
Equation (14) gives $P_0 = 35{,}000$ lb.
Equation (15) gives $d = 30.6''$ say 31″ $A_1 = 354$ sq.in.

In the above example the effect of the ducts is small ; the displacement of the neutral axis of the gross section is only 1/3 inch due to the presence of the ducts ; if the ducts had been ignored the depth would have been reduced 1 in. and the cable pull increased 10,000 lb. : these differences are small, due to the small cable eccentricity necessary with precast continuous spans, but the inclusion of the correction factors is necessary to guard against underestimating the stresses.

An example with similar spans, stresses and loading was given in the JOURNAL OF CONCRETE RESEARCH, No. 5, but this example had haunched beams with curved soffit, with mean area 350 sq. in., 29 in. deep at midspan and 43 in. deep at haunches, so that the uniform beam of 30 in. depth was 5 per cent. more economical in concrete, the cables were smaller, and there were no secondary stresses.

Haunched beams were economical when dead load predominated, e.g., for long spans principally.

It is clear (see Appendix 2) that for small beams subject to live loading haunches are not necessary, as the range of moment governs the dimensions, and this is not appreciably larger at the supports than at midspan.

It is also clear that the use of a preconceived cable profile is not advantageous and it is hoped that the above method and examples will be of service for the

stimation of the comparative economy of small span ontinuous beams, compared to simple spans, and will ssist in the design of such beams.

PART 2

Terminology

f = External applied moment at any section.
f_3 = Maximum range of moment at centre of span, due to superimposed load.
f_4 = Maximum external applied moment at any section.
f_5 = Maximum external applied moment at a support, or the range of moment at a support if this is larger, due to superimposed load.
f_6 = Maximum range of external moment at any cross section.

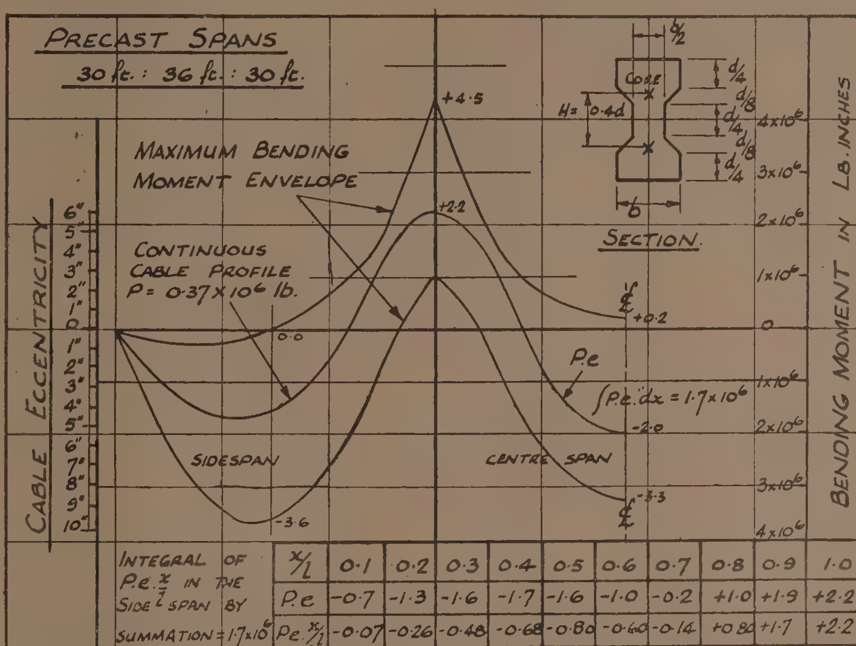
e_0 ; e_{01} = Maximum eccentricity possible for cables (always $+ve$).
H ; H_1 = Distance between " core points " of section.
h ; h_1 = Distance from neutral axis to core point opposite to cable duct.
h' ; h'_1 = D° D° in same side as cable duct.
(See also Part I, p. 261).

Appendix I

Example (4), using cables with Magnel's profile.

Magnel's cable profile will produce secondary " support " Moment in a uniform beam given by

$$M = \frac{e_2(.2l_1 + .234\ l_2) + .28\ e_4\ l_1 + .266\ e_3\ l_2}{\frac{l_1}{3} + \frac{l_2}{2}}\ P$$



x/l	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9	1·0
P.e	-0·7	-1·3	-1·6	-1·7	-1·6	-1·0	-0·2	+1·0	+1·9	+2·2
Pe.x/l	-0·07	-0·26	-0·48	-0·68	-0·80	-0·60	-0·14	+0·80	+1·7	+2·2

Fig. 9

L = Combined length of three continuous spans.
l = Length of central span of three continuous spans symmetrically arranged.
l_1 = Length of first span of a set of continuous spans.
l_2 = Length of second span of a set of continuous spans
x_1 = Distance from end support along first span.
x_2 = Distance along second span from third support.
P = Total pull in cables of prestressed beam.
P_0 = Cable tension required for the self weight of the beam of longest span, simply supported.
P_2 = Final continuous cable tension.
C = Maximum allowable compression stress.
ρ = Density of concrete $\left(\frac{1}{11.5}\ \text{lb./cubic inch}\right)$
w_0 = Uniform superimposed dead load.
w = Uniform superimposed live load.
W = Single point load.
b = Overall width of section.
d = Overall depth of section.

In the following symbols the suffix indicates that the dimension refers to the gross area of section neglecting the presence of cable ducts.

A ; A_1 = Area of cross section.
e ; e_1 = Eccentricity from neutral axis to centre of resultant pull in cables ($+ve$ above $N.A.$, $-ve$ below $N.A.$).

where e_4 : e_2 : e_3 $+ve$ upwards represent the cable eccentricity at centre of first span, at support, and at centre of centre span respectively. Hence for three equal spans

$$M = \frac{6}{5} P\ (.28\ e_4 + .434\ e_2 + .266\ e_3)$$
$$= P\ (.336\ e_4 + .521\ e_2 + .319\ e_3)$$

Hence for span ratios $\frac{2}{3}\ l : l : \frac{2}{3}\ l$

$$M = \frac{12}{13} P(.280\ e_4 + .551\ e_2 + .399\ e_3)$$

In the example No. (4), Fig. (7)

$P.e_4 = -14$, $P.e_2 = 43$, $P.e_3 = -33$

$\therefore\ M = +6.0$ tons ft.

This gives a 14 per cent. decrease in prestress moment at support and 18 per cent. increase at midspan, which is undesirable.

Appendix II

Moment Coefficients for Three Continuous Uniform Symmetrical Spans

w_0 = Uniform dead load, w = Uniform live load, W = Point live load. l = Length of central span

(a) Equal Spans

Moments at centre of Sidespan

Dead Load moment	$-.075\ w_0\ l^2$
Maximum live load moment	$+.025\ w\ l^2 + .04\ W\ l$
Minimum live load moment	$-.100\ w\ l^2 - .20\ W\ l$
∴ Range	$(.125\ w\ l^2 + .24\ W\ l)$

Moments at Support

Dead load moment	$+.10\ w_0\ l^2$
Maximum live load moment	$+.117\ w\ l^2 + .10\ W\ l$
Minimum live load moment	$-.017\ w\ l^2 - .03\ W\ l$
∴ Range	$(.134\ w\ l^2 + .13\ W\ l)$

Moments at Central Midspan

Dead load moment	$-.025\ w_0\ l^2$
Maximum live load moment	$+.050\ w\ l^2 + .04\ Wl$
Minimum live load moment	$-.075\ w\ l^2 - .18\ W\ l$
∴ Range	$(.125\ w\ l^2 + .22\ W\ l)$

(b) Span Ratios $\frac{5}{6} l : l : \frac{5}{6} l$

At Centre of Sidespan

Dead Load	$-.045\ w_0\ l^2$
Maximum live load	$+.027\ w\ l^2 + .04\ W\ l$
Minimum live load	$-.071\ w\ l^2 - .17\ W\ l$
∴ Range	$(.098\ w\ l^2 + .21\ W\ l)$

At Support

Dead load moment	$+.084\ w_0\ l^2$
Maximum live load moment	$+.096\ w\ l^2 + .09\ W\ l$
Minimum live load moment	$-.012\ w\ l^2 - .02\ W\ l$
∴ Range	$(.108\ w\ l^2 + .11\ W\ l)$

At Centre of Midspan

Dead load moment	$-.041\ w_0\ l^3$
Maximum live load moment	$+.031\ w\ l^2 + .03\ W\ l$
Minimum live load moment	$-.072\ w\ l^2 - .17\ W\ l$
∴ Range	$(.103\ w\ l^2 + .20\ W\ l)$

These ranges are lower than those for equal spans.

(c) Span Ratios $\frac{2}{3} l : l : \frac{2}{3} l$

At Centre of Sidespan

Dead load moment	$-.019\ w_0\ l^2$
Live load moment (maximum)	$+.028\ w\ l^2 + .047\ W\ l$
Live load moment (minimum)	$-.047\ w\ l^2 - .139\ W\ l$
∴ Range	$(.075\ w\ l^2 + .186\ W\ l)$

At Support

Dead load moment	$+.074\ w_0\ l^2$
Live load moment (maximum)	$+.081\ w\ l^2 + .094\ W\ l$
Live load moment (minimum)	$-.007\ w\ l^2 - .017\ W\ l$
∴ Range	$(.088\ w\ l^2 + .111\ W\ l)$

At Centre of Midspan

Dead load moment	$-0.050\ w_0\ l^2$
Live load moment (maximum)	$+.017\ w\ l^2 + .020\ W\ l$
Live load moment (minimum)	$-.067\ w\ l^2 - .163\ W\ l$
∴ Range	$(0.084\ w\ l^2 + .183\ W\ l)$

These ranges are lower than those for the span ratios $\frac{5}{6} l : l : \frac{5}{6} l$ and the ratios $\frac{2}{3} l : l : \frac{2}{3} l$ produce a more economical design if the centre span has a fixed value.

(d) Span Ratios $\frac{1}{2} l : l : \frac{1}{2} l$

At Centre of Sidespan

Dead Load Moment	$+.004\ w_0\ l^2$
Live Load Moment Max. $+ve$	$+.031\ w\ l^2 + .052\ W\ l$
Live Load Moment Max. $-ve$	$-.027\ w\ l^2 - .099\ W\ l$
∴ Range	$(.058\ w\ l^2 + .151\ W\ l)$

At Support

Dead Load Moment	$+.070\ w_0\ l^2$
Live Load Moment Max. $+ve$	$+.074\ w\ l^2 + .103\ W\ l$
Live Load Moment Max. $-ve$	$-.004\ w\ l^2 - .012\ W\ l$
∴ Range	$(.078\ w\ l^2 + .115\ W\ l)$

At Centre of Midspan

Dead Load Moment	$-.055\ w_0\ l^2$
Live Load Moment Max. $+ve$	$+.008\ w\ l^2 + .012\ W\ l$
Live Load Moment Max. $-ve$	$-.063\ w\ l^2 - .156\ W\ l$
∴ Range	$(.071\ w\ l^2 + .168\ W\ l)$

(e) Span Ratios $\frac{1}{3} l : l : \frac{1}{3} l$

At Centre of Sidespan

Dead Load Moment	$+.021\ w_0 l^2$
Live Load Moment Max. $+ve$	$+.034\ w\ l^2 + .021\ W\ l$
Live Load Moment Max. $-ve$	$-.013\ w\ l^2 - .065\ W\ l$
∴ Range	$(.047 + .086\ W\ l)$

At Support

Dead Load Moment	$+.071\ w_0\ l^2$
Live Load Moment Max. $+ve$	$+.072\ w\ l^2 + .114\ W\ l$
Live Load Moment Max. $-ve$	$-.001\ w\ l^2 - .007\ W\ l$
∴ Range	$(.073\ w\ l^2 + .121\ W\ l)$

At Centre of Midspan

Dead Load Moment	$-.054\ w\ l^2$
Live Load Moment Max. $+ve$	$+.003\ w\ l^2 + .006\ W\ l$
Live Load Moment Max. $-ve$	$-.057\ w\ l^2 - .148\ W\ l$
∴ Range	$(.060\ w\ l^2 + .154\ W\ l)$

An Investigation of the Behaviour of a Riveted Plate Girder under Load*

Discussion on Paper by S. Mackey, M.E., Ph.D., A.M.I.C.E., A.M.I.Struct.E., and D. M. Brotton, B.Sc., Ph.D. (Graduate)

In presenting the paper, Dr. S. Mackey expressed the author's regret that an arithmetical error involving the panel dimensions had occurred in Table 8 and was not discovered in time to alter the proofs. The authors had corrected this in the revised Table given below.

Table 8

Girder	Panel	Size h in.	Size d in.	Combination of stresses giving critical conditions. Tons/in.²	Tons/in.²	Theoretical critical load Tons	B.S.S. 449 Design shear stress tons/in.²
G 3	W. P 3	23¼	30	7.69	7.16	122	6.58
G 3	W P 8	17¼	30	10.70	10.74	171	10.28

Girder	Panel	B.S.S. 449 Design Load Tons	Elastic Behaviour Limit Tons	Ratio of Elastic Behaviour Limit of Theoretical Critical Load	Ultimate Load Tons	Ratio of Ultimate Load to Theoretical Critical Load
G 3	W P 3	104	160	1.31	199	1.63
G 3	W P 8	163	160	0.94	—	—

They would like to point out that in compiling this table the actual measured web thickness of 0.22 in. was used rather than the nominal thickness of 3/16 in., which accounted for the apparently high values of the B.S.S. 449 Design loads.

Dr. E. H. Bateman (Member), referring to calculated deflections, said he was always very surprised when any calculations which he did came out as near to the actual truth as the figures given in Table 2. He would like to know whether there was any mystery about the calculations or whether they were based on the type of simple formulæ which he had used in the old days when he had worked on a drawing board and had tried to estimate deflections. With triangulated girders it was possible to get somewhere near the truth if one knew the correct fudge factor to put in for the joints and for the effective lengths of the members, but when he saw results of calculated deflections coming out within one or two per cent. of the measured ones he wanted to be let into the secret of how to do it.

With regard to the lateral buckling due to the girder being supported on its side while being tested in a horizontal machine, he wished to know how the girder was supported in the machine, because presumably it could have been put on rollers in such a way that there would be no lateral bending.

In discussing the big diagonal buckle on the critical web member the author had made an interesting remark to the effect that the line of the buckle tended to pull round. That would have been a new idea to him had it not been for the fact that he had read Professor Pugsley's paper delivered to the Colston Society in Bristol in 1949, in which the Professor had presented an analytical explanation of the phenomenon. He asked whether the authors had tried to interpret their experience in the light of Professor Pugsley's theoretical solution.

One very small point he had noticed was that the authors had got extremely good value for their web thickness when they ordered 3/16ths and received .22. At a rough calculation it seemed the steelmaker was giving away 15 per cent., unless he weighed it all before sending it out. Perhaps the authors could say whether that was representative.

In regard to the crippling run it always seemed a great pity that one ever had to break anything because then it could not be used again, and on reading the paper he had wondered whether it might not have been possible for the authors to have put their heavy stiffeners in first and then to have loaded it until the strain gauges indicated that they were just running off the straight line shown in Fig. 14, taken that as the elastic limit for the heaviest of the stiffeners and then to have changed the stiffeners and got the corresponding figure for each series before the final break.

Mr. G. M. Boyd (Associate-Member) said he believed the experiment, very wisely, had been designed to investigate the behaviour of the webs, which was an extremely important unknown in structural engineering, and one point which had emerged was what was called, in the conventional literature, the critical buckling load of webs of that kind, did not seem to have any real meaning at all, and that the load which could be carried depended on the strength of the surrounding frame—in that case the two vertical stiffeners and the two horizontal flanges of the girder. The web itself could not actually cripple until those had given way, and research ought to move in the direction of studying the panels as a whole, including the surrounding or stiffening members.

As was done so often in those cases, the authors had congratulated themselves on the agreement between their experimental results and theory, but that agreement was really not at all surprising : if the theory was properly conceived there should be agreement. More emphasis ought to be directed towards studying the differences shown by the experiments as compared with the simple theories to which designers are accustomed. Such differences could be extremely important. The authors had given the average axial stress in the plate but had not mentioned much about surface stresses, yet the latter may be very high, without however, bringing about fracture. Designers rather tended to think in terms of fracture. For instance, the flanges of a girder were designed to withstand a certain stress, and it was often presumed that if it were carried to failure

*Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on April 24th, 1952. Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. (President), in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXX, No. 4, pp. 73-81 (April, 1952).

it would be by fracture of the flanges, but in practically all ordinary structures the failure was never by fracture unless the elusive and difficult phenomenon of notch brittleness intervened. In his opinion, the time was coming when designers should think more in terms of the actual modes of failure which were known to occur in structures, when they were considering factors of safety.

It was certainly rather a pity that the authors had been unable to eliminate lateral bending, as it complicated the picture very appreciably. Referring to the slide showing the distribution of stress across the flanges, in which there was a reduction on one side of one flange and a reduction on the opposite side of the other flange, in his opinion that was not explained simply by lateral bending but must imply torsion, and he would like the authors to elucidate the point a little.

The experiment had obviously been carried out with the intention of comparing the results with those obtained on the previously tested welded girder, and it would be interesting to have the authors' summing-up of the comparative behaviour of the two types of girder.

Mr. J. A. WILLIAMS (Member), referring to the point made by the first speaker with regard to the deflection of the girders, said he was rather alarmed by the closeness of the results between theoretical and experimental deflection, because he gathered the theoretical deflection had been worked out on the gross moment of inertia without any allowance for rivet holes and on the full value of $E = 13{,}450$. It was a fairly common drawing-office custom to make allowance first of all for the rivet holes in the tension flange, and also he had been taught to write in, rather euphemistically, "To allow for rivet slip take $E = 10{,}000$", and one arrived at what was assumed to be the correct deflection. He wondered whether the authors could give some indication as to how one ought to calculate deflection in order to get such nice results.

Mr. STANLEY VAUGHAN (Vice-President), said he was delighted to observe the close general agreement between theory and actual experimental results obtained.

For example, the average flange stress as calculated agreed closely with the average of the actual stresses measured at intervals across the flanges. There was, however, a curious variation in the measured stresses to which the authors had drawn attention, namely, that in the bottom flange there was considerable dip in stress intensity at a point about a quarter of the flange width at one edge and a corresponding dip on the other side of the upper flange, in a similar position but on the other side of the main axis. The authors had attributed these variations to secondary stresses caused by lateral bending due to the fact that the girder was placed horizontally and tested in that position. Secondary stresses due to this cause would however merely result in "tilting" the line of resultant stress—which would then vary from a maximum at one edge to a minimum at the other edge—and this would not account for the curious and appreciable dips in stress at points about a quarter of the way across the flange width.

It might be that this curous behaviour could be explained by the secondary effects due to torsion in addition to those due to lateral bending—both effects being of course superimposed on the main primary flange stress—but Mr. Vaughan would like to have the authors' further views on this question.

Finally, how comforting it was to find that even the very lightest stiffeners used in the tests gave the necessary stiffness to the web to enable it to behave properly as a web plate girder, and that in fact it made very little difference in the actual test whether the intermediate stiffeners were of the minimum size of 3 in. × 3 in. × $\frac{1}{4}$ in., or the maximum size of $4\frac{1}{2}$ in. × 3 in. × $\frac{1}{2}$ in. It would be interesting to know whether the authors had any proposals for designing intermediate stiffeners, because that was one of the points which had long caused a good deal of difference of opinion among design engineers.

Mr. BOYD asked Dr. Mackey how he accounted for the fact that the areas under the stress curves apparently did not add up. For example, the area in Fig. 4 did not appear to be equal to the area of the theoretical curve. He presumed this was another local bending effect.

Mr. BULLEN (Member of Council) said he noticed that the web plate was described as 36 in. deep and the distance between the flange angles was given as 36 in.: was it a fact that the web plate actually bore against the flange plates or were there the customary clearances?

Another point he wished to raise was with regard to the fitted stiffeners: was it really necessary with the intermediate stiffeners to fit them, or could they not just be laid flat against the web plate and not packed up?

Dr. BATEMAN asked whether the reason for fitting the stiffeners was not to provide support for the flange rather than the web.

Mr. BULLEN said that in the case of the main stiffeners it was desirable to ensure that the load was transmitted from the support into the main stiffeners, but in the intermediate stiffeners that condition was not required, and he thought it would have been sufficient, certainly in practice if not in the experiment, for the intermediate stiffeners merely to be laid against the web and not to be taken right up to the top flanges of the girder.

Dr. BATEMAN suggested that the stiffeners were brought up to support the flange angles against the tendency to buckle under compression. Due to the curvature of the girder as it deflected under load, the tendency to buckle was in the direction in which support was given by the stiffeners. Years ago his chief had made him spend a lot of time ensuring that stiffeners were fitted tight, and he believed it was the right method.

Mr. G. M. BOYD enquired whether there was any evidence of rivet slip during the experiment.

Mr. BULLEN asked whether it was essential to test the girders in the horizontal position or whether they could have been tested in the vertical position and the apparent complications avoided. From a quick perusal of the paper it appeared that not the weight of the girder but some torsional effects had given rise to the apparent complications.

Dr. MACKEY, in reply, said he would leave the bulk of the queries to be dealt with by Dr. Brotton, who had been responsible for most of the testing.

He was probably treading on very dangerous ground when speaking of the design of stiffeners but his own belief was that stiffeners on riveted work should be fitted to the compression flange but that it did not matter very much about the tension flange. That might be likened to the wisdom of King Solomon, in that he was going halfway to meet both his questioners on that subject, but he had mentioned in a previous discussion that such was his opinion and from the results since

obtained nothing had arisen to cause him to change his views.

With regard to the question of torsion, which had been raised by Mr. Boyd with reference to Fig. 4, the remarks made by Mr. Vaughan were very opportune. If the girder was bending upwards during the test there would be an added compression stress on the bottom half of the girder as tested which would increase the compressive stress on the compression flange on that side and correspondingly decrease the tensile stress on the lower half of the tension flange. Similarly if the girder was bending downwards the maximum reductions would occur in the opposite manner. As far as experimental work was concerned it was impossible to get a girder which was fabricated true to line, and in the present instance the initial web deflections confirmed this. They had more or less come to the conclusion, as in many research problems, that a process of elimination had to be adopted in order to come down to some hard and fast rules, and it was rather dangerous, from the results obtained on one particular riveted girder, to put forward recommendations for design, because the next girder they tested might give slightly different results.

The girder was tested in a 1,250-ton Avery testing machine of the horizontal type since no vertical testing machine capable of developing the ultimate failure load of 199 tons was available at the time. Financial considerations precluded the use of a complete set of roller bearings for supporting the girder in a horizontal position and hence greased runners were substituted as the best alternative method of support available. Vertical dial gauges bearing on the edges of the flange plates showed that the girder was not lying on all the runners, from which the authors had concluded that there was a certain amount of lateral bending involved. While it was quite possible that torsional effects may have contributed towards the stress depressions across the flanges shown in Fig. 4 there was no guarantee that at some point during the test one or other of the strain gauges might not have been functioning properly and hence the authors could give no definite conclusions on this matter.

The return of the experimental stress values to the theoretical values at the flange edges nearest the depressions was not influenced by any particular desire to make them return there, but was obtained from the recorded plots of the nearest strain gauges.

With regard to stiffeners, it was somewhat rash from the results of one test to put forward proposals for the design of intermediate stiffeners. The results however showed that even comparatively light section angles were adequate for this purpose and further tests of limited scope particularly applied to single plate girder panels would yield some valuable information on this topic.

The present paper did not cover all the work carried out in the investigation on plate girders, but the results obtained did not differ greatly from those obtained with the other girders of the series which were all of welded construction. Neither was it intended to convey the impression that the results obtained constituted all the information which could be derived from a single girder test. Time and expense had been limiting factors when the work was carried out and hence only selected measurements of strain and deflection were taken to reduce the time spent in testing to a minimum. It was felt that subsidiary tests on small specimens would cover most of the outstanding points and the authors were extremely grateful to Messrs. Dorman Long & Co., Ltd., for allowing so much time to be devoted to the testing of the large girders.

Dr. D. M. Brotton, referring to the lateral buckling due to the method of testing, said the girder was supported and loaded at three points, the four runners being in position underneath. If the flange plate was not exactly at right angles to the web plate that would tend to tilt the girder up a little and on applying the load there would be a tendency for the girder to lift. That led to a certain amount of lateral deflection and also a little torsion, as had been mentioned, and it probably accounted for the variations in the bending stress.

With regard to the deflection of the girder and a comparison of Fig. 3 with Table 2, the figures shown in the experimental deflection line in Table 2 were the actual experimental figures and they correspond to the extremity of the sum of the bending deflection and shear deflection.

As far as rivet slip was concerned, when the girder reached the crippling point they had kept well clear of it, not knowing quite what would happen, but no rivet heads flew off, and there was no sign of rivet slip evident.

Panel W.P.3 was the one which they had expected to fail, and they had focused their attention more on that than on any other. The load was limited by the deflection of that panel and not by the central deflection of the girder. During each loading run a deflection dial was placed against the centre of that panel, and it was watched during the application of load to see if it was increasing very rapidly, which would have indicated a failure coming along. At final failure the stresses on the underside of the girder reached a value something over 30 tons per square inch, so that even at 150 tons there had been a certain amount of surface yielding of the web panel. The web plate, however, yielded only on the surface and not right the way through and that area of local yielding had not affected the tests at all. This was repeated with the different stiffeners fitted, and when it came to the crippling run with the lightest stiffeners fitted, that area of yield extended, spread through the thickness of the plates, and then the panel itself collapsed. That was the ultimate cause of failure and the girder had been designed to do that.

The flange plate was not bearing on the top of the web plate, but was left just a little short. In regard to buckling of the compression flange as a whole if the edge of the flange plate was not supported it could buckle over the complete girder length. By putting intermediate stiffeners in and fitting them to the flanges it reduced the effective length of the flange plate which could then only buckle in between ; this considerably increased the strength of the compression flange. For the tension flange it did not matter so much because in this case it was not a question of instability.

Dr. S. Mackey said that in considering the close agreement between theoretical and experimental deflection it should be borne in mind that the test girder was loaded under almost ideal conditions. In practical girder work knife-edge loads and three point suspension occurred very rarely and consequently it was to be expected that the deflection readings obtained from practical girders would differ from those calculated on the theoretical assumptions. As Dr. Bateman had mentioned experiences in calculating deflections of triangulated girders he would refer him to some investigations which they had carried out on such girders, the results of which were being published in Vol. II of the Proceedings of the International Association for Bridge and Structural Engineering.

In reply to Mr. Boyd's query concerning the apparent lack of agreement between the areas under the experi-

mental and theoretical stress curves of Fig. 4 it should be pointed out that the effect of lateral bending would be to slope the theoretical stress curve which would make the agreement more obvious.

The CHAIRMAN pointed out that one question which had not been answered was that relating to the fudge factor ; it was a new expression, but he knew what the speaker meant, and it would have been interesting to have something from the authors on that subject.

With regard to the thicker plates, he did not know whether it was a new policy which had anything to do with the spending of public money or the giving of value for money, but it certainly might lead to arguments in practice. On the question of stiffeners, if the results of the one experiment were anything to go by, they seemed to indicate a considerable field for economy in the use of steel, which was very much in their minds these days.

Dr. MACKEY replied that he had avoided the question on fudge factor because he did not know what it meant, and he was still no wiser !

With regard to the thick plating, he would not like to convey the impression that designers could base their calculations in future on such an increased thickness. They had accepted it as 3/16 in. plating and when the results did not come out as expected they had measured it and found the differences mentioned. In deflection calculations made by the customary method one adopted a nominal thickness of material and if there were such big differences one could not hope to get accurate results. It was not so serious when the web was affected, but if there was, say, a 15 per cent. reduction in the flange thickness it would seriously affect the agreement between theory and practice. Apart from this it did not follow that the discrepancy would be uniform throughout the material. In rolling mild steel plate, springing of the rolls might tend to give increased plate thickness at the centre of the plate as rolled, compared with that obtained at the edges. In view of the uncertainty of this, however, no reliance could be placed on it for design purposes.

Finally, in expressing gratitude on behalf of Dr. Brotton and himself for the very cordial way in which they had been received, he observed that most of the questions had appeared to cast doubt upon the accuracy of the relative results of theoretical and experimental readings. With uniform material and the adoption of the measured value of E obtained from test pieces cut from the actual test member, it was possible to obtain experimental deflection readings agreeing very closely with the theoretical values.

The proceedings then terminated.

Written Communications

From Mr. K. C. ROCKEY : The authors have stated that they were unable to use Southwell's method for determining the experimental buckling loads from the load-lateral deflection graphs since these were not hyperbolic in form. This problem of determining the buckling load from experimental results is a difficult one, and in the writer's experience one is rarely able to use Southwell's method with any success on shear panels. One method which the writer has found to give reasonable results is that of plotting graphs of applied load against the axial load in vertical stiffeners. Prior to buckling the load in the vertical stiffeners should be zero or very small, since the shear force is carried by a normal shear action. However, after the plate has buckled the load in the vertical stiffener increases since part of the applied shear load is now carried by a truss action. Thus at a load corresponding to the buckling load of the panel there should be a "knee" on the graph, marking where the slope of the graph decreases. Since this "knee" is usually well pronounced no difficulty is obtained in measuring this buckling load.

The wave formations plotted in Fig. 5 are very interesting since they show, in a very simple manner, how the presence of applied bending stresses acting with a given shear stress will lower the buckling shear stress of a panel below that value obtained for the case of pure shear. Thus, although the two similar panels *WP*1 and *WP*3 are subjected to the same shear stress, the depth of the main buckle in panel *WP*3 is four times the depth of the buckles in panel *WP*1, thus indicating that the buckling load of panel *WP*3, which is subjected to a high bending stress as well as the shear stress, is lower than the buckling load of panel *WP*1.

The lateral deflection curves for panel *WP*5, shown in Figs. 9 and 10, are typical profile curves for panels buckled under the action of pure bending stresses and from the definite wave formation shown it would look as if buckling of the panel was taking place, although the load was below the theoretical buckling load.

With reference to the use of five sets of vertical stiffeners in an attempt to determine the effects of stiffener rigidity upon the behaviour of the girder, it is noted that even the lightest stiffener possessed a flexural rigidity many times that required by theory, and should therefore provide a very effective support to the web-plate. That this was so has been stated in the paper and explains why no relationship existed between the depth of the buckles in the panels and the rigidity of the stiffeners.

The flanges of this riveted girder are very flexible and there are a number of statements in the paper which would appear to indicate that this flange flexibility has influenced the behaviour of the webplates. For example, in the section dealing with the crippling run on the girder, reference is made to the fact that at a load of 160 tons a change in the shape of the buckles in the panel took place, whilst later in the paper reference is made to the fact that as the maximum load was approached the major buckle in each panel oriented itself so that it finally lay exactly along the tension diagonal of the panel. Both of these statements seem to indicate that the flange was deflecting under the lateral load imposed by the web, thus allowing the buckle to shift in the panel.

The writer, in collaboration with Mr. H. J. M. Watkins, B.Sc., is investigating the effects of flange flexibility upon the behaviour of the webplates of plate girders, and so far the results obtained have indicated that it is desirable that girders should possess flanges having a minimum flange flexibility factor (I/b^3t) of 0.005 ; where I is the moment of inertia of the flange about its centroid, b the stiffener spacing and t the thickness of the webplate. If a lower I/b^3t value is accepted, then deep buckles will be formed and high membrane stresses will be set up. The I/b^3t value of 0.002 for the riveted girder is considered to be rather low.

In the final section of the paper the authors have compared the deflections obtained with the welded and riveted girders. However, since the I/b^3t value for the welded girder was 0.007 as compared with the value of 0.002 for the riveted girder, care must be exercised in making any comparison on the magnitudes of webplate deflections at loads above the buckling load.

Finally, the writer would like to congratulate the authors on their very fine paper.

From Dr. H. Gottfeldt (Member) : In the course of the discussion a question was put to the authors which arises almost invariably whenever plate girders are under consideration, namely, whether the web stiffeners should be fitted, or even connected, to the flanges.

While admiring the "Solomonity" of the authors' reply that stiffeners should be fitted to the compression flanges but not to the tension flanges, I cannot agree with their reasons for this judgement.

If the stiffeners are intended to prevent the compression flange from buckling then their space ought to be related to the force, shape, and the dimensions of this flange, but I am not aware of any such regulation. Neither am I aware of any reported failures of plate girders where the failure has been attributed to the lack of fit between stiffeners and flange.

What does this supposed danger of buckling of the compression flange really mean? Does it refer to lateral instability of the flange as a whole? This is safeguarded against, e.g., in the case of a through bridge, by the stiffeners being suitably braced against the cross-girders and a sufficient width of flange between the panel points.

Or is a sort of torsional failure envisaged, with the edges of the compression flange tending alternately to move up or down? This would imply bending of the web out of its plane, and if this danger exists at all the most dangerous point would be in the web just outside the flange angles. This point is safeguarded by the ordinary stiffeners being connected to the vertical legs of the flange angles and the suggested fitting of the flanges would have no further influence on this.

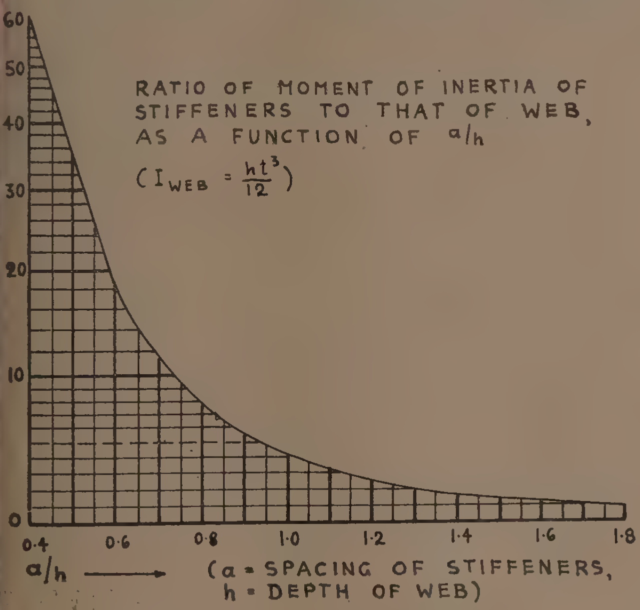


Fig. 1

Or is a local buckling of the *edges* of the flange plates to be feared? In this case the stiffeners should not only be fitted but also connected to these edges as they would be just as likely to buckle upwards as downwards, and once more their spacing ought to be a function of the width and thickness of the plate.

I can think of no other possible modes of buckling, and none of these seems to call for the suggested fit.

The cost of fitting the stiffeners to the flanges is probably small, and from an economic point this matter is therefore of no great importance. One that is was pointed out by the President when he drew attention to the fact that the *size* of the stiffeners did not seem to have any appreciable effect on the capacity of the girder.

The question of the required size—or stiffness—of the stiffeners has been tackled by Timoshenko[1], and the writer tried some years ago to popularise his results by means of a graph[2] from which the required moment of inertia of the stiffeners can be obtained as a function of the spacing of the stiffeners and of the moment of inertia of the web (for the weak axis).

This graph is shown in Fig. 1 and it will be seen that the size of the stiffeners decreases with their distance. In other words, the thicker the web the wider apart *and* the smaller the stiffeners, a result that agrees much better with common sense than the rule of thumb in B.S.S. 449, clause 47 (a).

For the plate girder under test the moment of inertia of the web is $36 \times 0.1875^3/12 = 81/256$ in.^{4}, the ratio of the spacing of the stiffeners to depth of web is about 2/3, and the numerical value obtained from the graph is about 12. The required moment of inertia of the stiffeners is therefore $12 \times 81/256 = 3.8$ in.^{4}.

The smallest stiffeners used by the authors have a moment of inertia of 4.85 in.^{4} and are therefore still well above the value obtained from the graph, although, according to the B.S.S. rule the outstanding leg should be at least $36/30 + 2 =$, say, $3\frac{1}{4}$ in. instead of the 3 in. leg used by the authors.

I therefore concur with Mr. Andrews' remark that the stiffeners tend to be over-dimensioned and that there is here a potential source of saving money and steel, if only the B.S.S. rule were replaced by some more rational law.

From Dr. A. R. Gent : I have recently been investigating the torsional properties of riveted members and should like to make a few remarks concerning the torsional results given in the paper. My conclusions have been based on the results of tests on full-scale members fabricated from flat plates face to face, which represent the fundamental components of an I-section, and variation of rivet pitch and breadth have been considered. Although I have not had the opportunity of testing a complete member of this form, I can add a few remarks about their general behaviour and shall endeavour to show this single result in its true perspective.

Firstly, concerning the actual stiffness values, the increase of stiffness with the use of heavier stiffeners is to be expected since they were bolted directly to the web, thus increasing the effective web thickness over short lengths of the member. The outstanding leg should not, however, affect the stiffness and the result for stiffeners (3) and (4) should be equal, as is approximately the case. Within this degree of accuracy, i.e., approximately 2 per cent., the result for stiffeners No. (5) can be considered the same and it is likely that this value of about 9.5 in.^{4} represents the true stiffness of the girder which would have been obtained without the intermediate stiffeners : this is probably because the bolts will only provide effective frictional restraint between the web and the stiffeners over a small area when thin plate is used. The increase in the stiffness above this value due to the $\frac{3}{8}$ and $\frac{1}{2}$ in. stiffeners is higher than that given by approximate calculations but this effect is not of appreciable importance.

The low value of 9.14 in.^{4} given in the paper for stiffness calculated by relaxation methods assuming the section acted as solid was surprising compared with the basic experimental value of 9.5 in.^{4} and led to an investigation of this result. An approximate check using the graphs given by Dr. Dobie in his recent paper to the

[1]Timoshenko. "Theory of Elastic Stability," 1936. 417 pp.
[2]"Welding." February, 1945.

Institution ("The Torsional Strength of Structural Members," THE STRUCTURAL ENGINEER, Vol. XXX, No. 2, 1952), with a slight modification to suit the special shape considered, gave a value of 11.5 in.^{4} and a further check using a relaxation net covering the whole section for which the errors were reduced to below 1 per cent. gave a value of 10.7 in.^{4}. This latter result is unlikely to be in appreciable error and is as expected greater than the experimental value which will have been adversely affected by slip between the components.

General conclusions regarding the stiffness of riveted members should not be drawn from this single result obtained over a very small range of loading, i.e., 0 to 3.65 tons in. (I am indebted to the authors for this extra information) which only represents a deflection of the order of 0.7×10^{-4} rad/in. as tests on sections over a greater range of twists show an appreciable decrease of stiffness with loading which can be related to the equivalent shearing loads on the rivets. As the frictional resistance between the components is gradually overcome the stiffness decreases from a value approaching that of the equivalent solid section and the rate of decrease is dependent on the rivet spacing. For this section with close rivet spacing, this decrease should not be appreciable within the working range and the value of 9.5 in.^{4} would probably be satisfactory, however with lighter riveting a more conservative stiffness would have to be adopted in calculations.

Authors' Reply to Written Communications

The authors were very grateful to Mr. Rockey for his valuable comments. Although on a few occasions they had been able to use Southwell's method satisfactorily for estimating web buckling loads these were exceptional cases and it was gratifying to them to learn that other research workers had experienced similar difficulty. The method suggested by Mr. Rockey for estimating the web buckling load from the graph of applied load *versus* stiffener load gives fairly reasonable results but is not as accurate as that obtained from a graph of web deflection against applied load as shown in Fig. 16. This is particularly evident where the total stiffener load up to the point where truss action takes place is very small. In such cases the presence of bending stresses in the stiffeners may completely mask the direct stresses and lead to erroneous estimation of the stiffener load. The issue is further complicated by the gradual alteration to the web panel contours from the initial to the buckled state. Any change of this nature must affect that portion of the web taken as acting with the stiffener angles, rendering it more or less effective, depending on the type of change developed. The authors had not considered the effect of the flange flexibility factor on web buckling and are grateful to Mr. Rockey for drawing their attention to this. It is to be hoped that in due course Mr. Rockey and Mr. Watkins will publish a complete record of their findings on this aspect of plate girder design.

Dr. Gottfeldt's contribution to the discussion deals very fully with the restraining effect of the stiffeners on flange buckling. It should be borne in mind, however, that the main purpose of intermediate stiffeners is to stabilise the web plate and to prevent buckles from spreading from one web panel to another, which would have the effect of increasing the effective panel size and thereby reducing the critical load. In design problems the size and spacing of the intermediate stiffeners should primarily be determined from consideration of the above factors and, as pointed out by Dr. Gottfeldt, there is a definite need for a more rational approach to stiffener design. While the authors are in general agreement with the arguments put forward by Dr. Gottfeldt against fitting the intermediate stiffeners to the flanges, they still prefer to see them fitted to the compression flange. In the girder under review the edges of the compression flange tended to buckle downwards rather than upwards. A similar tendency was observed with the welded girders of the series and in the absence of evidence of upward buckling, it is felt that the value of intermediate stiffeners will be increased when they are fitted to the compression flange. At loads approaching the ultimate girder load when truss action is taking place the additional end bearing area at the compression flange will also counteract the possibility of overloading the rivets through the stiffeners and compression flange angles which may arise from web buckling.

Dr. Gent has raised some interesting points regarding the torsional resistance of the girder. The torsion test was carried out as an ancillary to the main investigation and was not therefore carried to a degree of twist which might influence the behaviour of the girder in the subsequent tests.

The graphs in Dr. Dobie's paper referred to by Dr. Gent were derived mainly from tests on rolled beams and would be expected to give conservative results when applied to plate girders. In calculating the torsion constant by relaxation methods the authors did not carry the calculations as far as Dr. Gent, which may account for the difference between his value and that given in the paper. The rivet pitch used in the girder was not greatly in excess of that which would be expected to give integral behaviour and the authors concur with Dr. Gent that the decrease in torsional stiffness due to separate behaviour would be slight.

Book Review

The Design of Prismatic Structures, by A. J. Ashdown, A.M.I.Struct.E. (Concrete Publications, Ltd., 1951. $9\frac{1}{2}$ in. $\times$ $6\frac{1}{4}$ in., 65 pp. 8s.)

This small volume of 65 pages of text will be a valuable addition to the many publications on specialised structural topics issued by Concrete Publications, Ltd., in the well-known blue binding. Their excellent standards of presentation are well maintained with clear diagrams of ample scale. The book contains four chapters: Prismatic Structures of One Span, Multiple-Bay Structures, Continuous Prismatic Structures and Prismatic Structures with Sloping Ends. The first deals with the principles used in the analysis of the structures stating the approximations which have been made; this chapter requires some application in order to grasp its essentials. The book is evidently written for structural engineers of adequate theoretical background and introduces a number of analytical procedures such as the "moment-balance" method, "column analogy," etc., for the design of the continuous slabs and fixed-ended stiffening ribs, but in each case one reference is given to published information of use in further study. Little information is given regarding practical details and in this connection readers might wish to know of the description given of a roof of this type in CONCRETE AND CONSTRUCTIONAL ENGINEERING of January, 1952.

The book will be found of value to all those interested in these modern types of structures. R. J. W.

Institution Notices and Proceedings

MACLACHLAN LECTURE

The MacLachlan Lecture for 1952 will be given at 11, Upper Belgrave Street, London, S.W.1, at 6.0 p.m. on Thursday, November 13th, 1952, by Mr. L. E. Ward (Associate-Member). The Lecture will be entitled "The Design and Construction of a Three-Bay Aluminium Aircraft Hangar at London Airport."

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, November 27th, 1952

Ordinary General Meeting at 5.55 p.m. This meeting is for the election of Members and is open only to corporate members of the Institution. It will be followed at 6.0 p.m. by a Joint Meeting with the British Section of the Societe des Ingenieurs Civils de France, when Monsieur I. Leviant will give a paper entitled "An Introduction to Vacuum Concrete."

Thursday, December 11th, 1952

Ordinary General Meeting for the election of Members at 5.55 p.m., followed by an Ordinary Meeting at 6.0 p.m. Mr. Donovan H. Lee, B.Sc., M.I.C.E., M.I.Mech.E., M.Am.Soc.C.E. (Member of Council), will give a paper on "Prestressed Concrete Bridges and other Structures."

Thursday, December 18th, 1952

Ordinary General Meeting for the election of Members.

Thursday, January 8th, 1953

Joint Meeting with the Reinforced Concrete Association at 6.0 p.m., when Colonel A. R. Mais, O.B.E., T.D., and Mr. A. C. Little will give a paper on "The Construction of Eight Prestressed Concrete Tanks."

Thursday, January 22nd, 1953

Ordinary General Meeting for the election of Members at 5.55 p.m., followed by an Ordinary Meeting at 6.0 p.m., when Mr. B. A. E. Hiley, M.I.C.E. (Member of Council), will give a paper on "Electricity Generating Stations."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1953

The Examinations of the Institution will next be held at Centres in the United Kingdom and overseas on January 6th and 7th, 1953 (Graduateship), and January 8th and 9th, 1953 (Associate-Membership).

EXAMINATION PASS LIST. JULY, 1952

Home Centres

The Examinations were held in July, 1952, at the usual centres in Great Britain.

One hundred and eight candidates took the Graduateship Examination, and 292 took the Associate-Membership Examination, making a total of 400. Of these, 66 passed the Graduateship Examination, and 112 passed the Associate-Membership Examination.

The names of the successful candidates are :—

Graduateship Examination

Adams, Peter Henry, Attwood, Roy Leonard, Bailey, John, Baker, Derek William, Ball, Thomas Christopher Gann, Barlow, Eric, Barnes, Victor, Barr, James Anthony, Bateman, Douglas James, Capelin, Dennis Edwin, Chilvers, Alan, Clarkson, Reginald George, Coleman, Peter Stanley, Conway, Gerald Ernest, Czarnecki, Jan, Dampier, Bernard William, De Penning, Jean Charles, Durley, Anthony William, Earp, Clifford David, Ellis, Kenneth George, Evans, Robert Edgar, Everett, Frank Joseph, Farley, Michael George Gilbert, Finch, Cecil William, Gach, Jerzy, Greene, Dennis Leslie, Haden, Godfrey Eric, Holmes, Gordon Victor, Holocher, Roman Wieslaw, Iwanski, Zygmunt, Janka, Kazimierz, Jenkins, Roy George Henry, Jones, Royston William, Kirkman, Cyril John, Krol, Tadeusz, Kucharski, Albin Michael, Lenartowicz, Witold, Lewis, Brian David, Llewellyn Peter Charles, McNally, Eneas, Mason, John Francis, Masters, Patrick Anthony, Musgrave, Denis, Noble, Colin, Northgreaves, Kenneth Roy, Nuttall, Peter, Orpiszak, Boleslaw, Phillips, William, Roberts, Geoffrey Russell, Rose, Charles Frederick, Rzadkiewicz, Stanislaw, Shotton, James Edward, Slowikowski, Leszek, Smith, Jerrold Arthur Joseph, Stacey, Donald William, Stanyon, Philip George, Strachan, Alan, Studzinski, Stanislaw Henryk, Thei, Arthur Abraham, Wadon, Jan Jerzy, Walker, Geoffrey, Walker, Sidney, Warriner, Harold Maurice, Whitesmith, Noel Gilbert, Wilkes, Brian Edward, Wotton, Frederick Ernest.

Associate-Membership Examination

Anderson, Robert, Athavale, Shrikrishna Gangadhar, Bailey, Stanley, Barcham, William John, Barnes, Robert Gerald, Baron, John William, Bartak, Andrzej Josef Jerzy, Bartle, Peter Ronald, Bayley, Jack Borough, Bennett, Colin Joseph, Bennett, Robert Walter, Billington, Roger Dean, Binns, Robert Derek, Black, Robert, Bletcher, Raymond Leonard, Bond, Peter Henry, Booth, Robert Stuart, Bregosz, Stanislaw, Brough, Derek Fletcher, Bryant, Michael Ernest, Burslem, John Austin, Campbell, Gordon Arnison, Campbell, Peter Leonard, Canning, Philip John Alexander, Carolan, Edward Joseph, Chapman, Kenneth Geoffrey, Chappell, Donald, Chappell, Ronald Charles, Clark, George Hyde, Creet, John George, Crosthwaite, Donald Rothery, Crutchley, Leslie, Cussons, Stanley Harold, Davey, Kenneth Clive, Day, James Archie, Dembinski, Maciej Jerzy Josef, Dennard, Bertram, De Souza, Ronald Cajetan, Duncan, John, Edwards, Jack Alfred, Edwards, Philip Birchall, Ellis, Ryan, Elsey, Morris Burfield, Evans, David Edmund, Eyres, Derek, Farnaby, John Eric, Foster, Rodney, Frischmann, Wilem William, Frost, Lewis Lake, Fuller, Edward Thomas, Gawlinski, Andrzej Wilhelm, Gorecki, Alexander, Halsall, John Denys, Halstead, Donald Harris, Harris, David William, Harrison, Edward, Harvey, John Alan, Harvey, Wreyford Frank Petrie, Hicks, Edward George, Hill, John Worsley, Hill, Peter Henry, Hogan, Eugene Peter, Jarvis, Anthony Peter, Jenkins, Robert William, Jones, Ivor Philip Thompson, Kadiani, Fida Husein Ghulamali, Kay, Marzell, Kinder, Graham, Landau, Robert Elkan, Langford, Lewis, Lee, Gerald Stanley, L'Oste-Brown, Anthony Joseph, McCarte, John, McCauley, John Brian, McGregor, David Hugh, Maillardet, Roy, Male, Edwin Talbot, Mann, Frank Thomas, Martin, William George, Morray-Jones, Robert Henry, Nabi, Gamil Abdel, Naylor, Gerald

Wilfrid, NOHR, Max, NORFOLK, John Duncan, NORMAN, John Leonard, OVENS, Hubert, PAGE, Frank Arthur, ROBINSON, William Crathern, ROGERS, Kenneth Jordan, ROWLAND, Vernon Roy, ROWLEY, Noel, SALTER, Terence Herbert, SHIELDS, Douglas John, SMITH, Peter Frederick, SMITH, Reginald George, SNOWDEN, George William, STAIG, Alan Franklin, SUGDEN, Derek Taylor, SUNNAK, Sardari Lal, TANNER, Peter Christopher, TAYLOR, Gerald, THOMPSON, Peter Charles, THORELY, Walter, TINSLEY, Peter Hugh, TOWLER, Robert Jack, WADDY, Harry Francis, WEBSTER, Donald Kenneth, WEEKS, Peter Charles, WELLS, Kenneth James, WILLIAMS, Keith Henry Glyn, WOOD, Peter Buckley, WOODS, Malcolm Derek.

MACLACHLAN LECTURE COMPETITION, 1953

The closing date for the receipt of entries for the next MacLachlan Lecture Competition will be Tuesday, March 31st, 1953. Particulars of the Competition are as follows :—

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering so long as in every second year the subject shall be confined to steel structures. (1953 is one of these years.)

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s. 0d.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer these sums to the Research Fund of the Institution.

PARTICULARS OF THE COMPETITION FOR 1953

1. The MacLachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1953.

2. The subject of the lecture shall be confined to steel structures.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Tuesday, March 31st, 1953.

DRURY MEDAL AWARD

The fourth competition for the above award will take place in 1953. The subject is the design of the structure of a new factory building. The material of construction is entirely at the choice of the competitor. The competition has been designed to encourage ingenuity of structural arrangement. Economy in the use of steel is an important feature of this year's competition.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary ; envelopes to be marked in the top left-hand corner, "Drury Medal Award."

The closing date for the competition is October 1st, 1953.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

HONOURS AND AWARDS

In offering their sincere congratulations to the following member on the distinction recently conferred upon him, the Council feel that they are also expressing the good wishes of the Institution.

Order of the British Empire—O.B.E.
Mr. R. A. HATFIELD (Associate-Member).

PERSONAL

Mr. P. B. Steer (Associate-Member) has joined Messrs. Stoyer & Adcock, Consulting Engineers, of 122, Wilton Road, London, S.W.1, as a partner.

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will take place at 11, Upper Belgrave Street, London, S.W.1, on Tuesday, November 11th, 1952, at 6.0 p.m., when the following films will be shown by courtesy of the British Iron and Steel Federation :—

(a) "Great Achievement."
(b) "Steel in West Yorkshire."
(c) "Steel."
(d) "River of Steel."

Hon. Secretary : C. Allen Brown, 43, Coolgardie Avenue, Highams Park, London, E.4.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Wednesday, November 19th, 1952

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator), on "Unusual Design for a Large Constructional Shop."

Tuesday, January 13th, 1953

Joint Meeting with the Institute of Welding, Liverpool and District Branch, when the 1951 Larke Medal Paper on "Continuous Welded Structures, Abbey Works, Port Talbot," will be given by W. S. Atkins, B.Sc., M.I.C.E., M.Inst.W., at the Liverpool College of Technology, at 7.0 p.m.

Wednesday, January 28th, 1953

Mr. Ronald Oates (Graduate), on "The Structural Design of the Mediæval Cathedral."

All Meetings, unless otherwise stated, will be held in the Reynolds Hall, College of Technology, Manchester, at 6.30 p.m., preceded by tea at 5.45 p.m.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 24, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, November 18th, 1952

Film : "Fawley Refinery," Parts 1 and 2, at Derby, at 7.0 p.m.

Friday, November 28th, 1952

Mr. E. McMinn, on "Tubular Structures."

Friday, January 23rd, 1953

Dr. K. Hajnal-Konyi, A.M.I.C.E. (Member), on "Recent Applications of Shell Concrete Construction in England and Wales."

All Meetings, except where otherwise stated, to be held in the James Watt Memorial Institute, Birmingham, at 6.0 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES BRANCH—GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Wednesday, November 26th, 1952

Joint Meeting with the Student Section of the Midlands Association of the Institution of Civil Engineers, Mr. E. T. Edwards, F.G.S., A.M.I.Mech.E., on "Boreholes in the Midlands for Water Supply and Foundations," at Birmingham Civic Centre (Room 129), at 6.30 p.m.

Friday, January 30th, 1953

"Further Recent Midland Structures." Descriptions of Rebuilding of Marshall & Snelgroves and C. & A. Modes (War Damage), Birmingham Technical College (Steel Frame Building), and Grosvenor House, Birmingham (Foundations), at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 7.0 p.m.

Hon. Secretary : F. G. Fletcher, 60, Brean Avenue, South Yardley, Birmingham, 26.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, November 4th, 1952

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator), on "Unusual Design for a Large Constructional Shop," at Middlesbrough.

Wednesday, November 5th, 1952

The above meeting will be repeated at Newcastle.

Tuesday, December 2nd, 1952

Mr. D. M. Brotton, B.Sc., Ph.D. (Graduate), on "Relaxation Methods" at Middlesbrough.

Wednesday, December 3rd, 1952

The above meeting will be repeated at Newcastle.

Tuesday, January 6th, 1953

Joint Meeting with the Institution of Civil Engineers at Middlesbrough.

Wednesday, January 15th, 1953

Joint Meeting with the Northern Architectural Association, at Newcastle.

All meetings will commence at 6.30 p.m., the Middlesbrough meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and the Newcastle Meetings in the Neville Hall, near the Central Station.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, November 4th, 1952

Mr. Stanley Marchant, B.Sc., A.M.I.Mech.E., on "Theory and Practice of Prestressed Concrete."

Tuesday, December 9th, 1952

Films : "Welded Structures," and "New Tyne Bridge"—kindly lent by Messrs. Dorman Long & Co., Ltd.

Monday, January 19th, 1953

Joint Meeting with the Institution of Civil Engineers, Northern Ireland Association. Mr. Harold E. Sidwell, M.Sc., A.M.I.C.E., on "Reinforced Concrete Building in Brazil," at Queen's University, Belfast, at 5.45 p.m.

All meetings, unless otherwise stated, will be held at the College of Technology, Belfast, at 6.45 p.m., preceded by tea at the Overseas League Premises, Wellington Place, Belfast, at 6.0 p.m.

Hon. Secretary : S. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Tuesday, November 18th, 1952

Mr. E. McMinn, on "Tubular Structures," at the Ca'doro Restaurant, Glasgow, at 6.0 p.m.

Tuesday, December 16th, 1952

Mr. J. H. Huntley, on "Structural Design of Cranes," at the Ca'doro Restaurant, Glasgow, at 6.0 p.m.

Hon. Secretary : D. G. Drummond, B.Sc., M.I.Struct.E., A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held at the Duke of Cornwall Hotel, Millbay, Plymouth, on Wednesday, November 5th, 1952, at 7.0 p.m. The President and the Secretary of the Institution will attend the Meeting.

The following meeting has been arranged :—

Friday, January 23rd, 1953

Mr. Leslie Richardson, A.M.I.C.E. (Associate-Member) on " Construction of Two Power Stations in the South-West," at the Duke of Cornwall Hotel, Millbay, Plymouth, at 7.0 p.m.

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10-12, Market Street, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Saturday, November 1st, 1952

The Chairman's Address, which was given by Colonel R. D. Heseltine, T.D., D.L. (Member), at Cardiff, on October 17th, will be repeated at Colwyn Bay, and will be followed by a discussion.

Thursday, November 13th, 1952

Mr. R. G. Braithwaite, M.I.C.E., on " Electric Screw Piling," at Cardiff.

Wednesday, November 19th, 1952

The above meeting will be repeated at Swansea.

Monday, December 1st, 1952

A meeting will be held at Cardiff at which films will be shown.

Wednesday, December 3rd, 1952

A meeting will be held at Swansea at which films will be shown.

Wednesday, January 21st, 1953

Junior Members' Evening at Swansea.

Meetings in Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings in Swansea will be held at the Mackworth Hotel, at 6.30 p.m.

Meetings in Colwyn Bay will be held at the County Buildings at 6.0 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, November 7th, 1952

Combined Meeting with the Institution of Civil Engineers. Mr. G. P. Bridges, A.M.I.C.E., L.R.I.B.A. (Member), on " The Design and Construction of Reinforced Concrete Silos and Bunkers."

Friday, December 5th, 1952

Mr. P. J. Ward, on " The Design and Erection of Television Masts."

Friday, January 2nd, 1953

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator), on " Unusual Design for a Large Constructional Shop."

All meetings will be held in the University of Bristol Geology Lecture Theatre at 6.0 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 39, Effingham Road, St. Andrew's Park, Bristol, 6.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, November 19th, 1952

Mr. G. C. Cummings, B.Sc., on " Concrete Grain Silos at Louth."

Wednesday, December 17th, 1952

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator), on " Unusual Design for a Large Constructional Shop."

Wednesday, January 21st, 1952

Mr. Donovan H. Lee, B.Sc., M.I.C.E., M.I.Mech.E. (Member of Council), on " Design of Prestressed Concrete."

All meetings will be held at the University, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

ADDITIONS TO THE LIBRARY

TSCHEBOTARIOFF, G. P. *Soil Mechanics, Foundations and Earth Structures ; An Introduction to the Theory and Practice of Design and Construction.* New York and London, 1951. Presented by Mr. A. C. Hobbs.

GEDDES, Spence. *Building and Civil Engineering Plant : Its Purchase, Application and Operation.* London, 1951. Presented by Mr. D. F. Weare.

Proceedings of the American Society for Testing Materials, Vol. 51, 1951. Philadelphia, 1952.

WHITLAM, E. F. *Healing in Concrete after Compression Failure or Autogenous Healing* (Thesis submitted for the Degree of Master of Science, University of London, 1950). Presented by the Author.

FABER, Oscar. *Reinforced Concrete.* London, 1952. Two copies, presented by Professor A. L. L. Baker and Mr. L. E. Hawkins.

KOCH, J. J., and Others. *Strain Gauges : Theory and Application.* London, 1952. Presented by Dr. D. M. Brotton.

LANGHAAR, J. L. *Dimensional Analysis and Theory of Models.* New York, 1951. Presented by Dr. S. Mackey.

ASHDOWN, A. J. *The Design of Prismatic Structures.* London, 1951. Presented by Mr. R. J. Wilkins.

STEWART, D. A. *The Design and Placing of High Quality Concrete.* London, 1951. Presented by Mr. W. Hunter Rose.

SCOTT, W. Basil. *Steelwork in Buildings : A Commentary on the B.S.S. on the Use of Structural Steel in Building.* London, 1952. Presented by the Author.

THIRLWELL, J. B. *Strength of Materials*, London, 1952. Presented by Mr. C. A. R. Eslick.

PRAGER, W. and HODGE, P. G., Jr. *Theory of Perfectly Plastic Solids.* New York, 1951. Presented by Dr. A. R. Gent.

MERIAM, J. L. *Mechanics : Part I—Statics.* New York, 1952. Presented by Mr. F. Hyde Blake.

BREND, H. J. *Means of Escape in Case of Fire.* London, 1952. Presented by Mr. A. H. Ley.

Presidential Address*

By E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E.

I wish to thank the members of the Institution most sincerely for the honour they have conferred on me in electing me President for the coming year. I am deeply sensible of the honour and responsibility of presiding over this vigorous Institution of over 6,000 members and while I am very conscious of my many shortcomings, I am also fully aware that I shall receive the unstinted support of your Council, the Secretary and the staff of the Institution. During my term as President it will be my constant endeavour to further the aims and objects of the Institution as set out in our Charter.

It is not unusual for the President in his address to refer to the work or town with which he has been associated during his professional career, and therefore my first thoughts when considering this address were in connection with London. It soon became apparent however, that the subject of structural engineering works in London was so vast that it would be necessary to confine my remarks to a specific part of the metropolis. I shall therefore describe, sometimes very briefly, works in connection with the river Thames, which have been essential for the growth and development of London and shall generally confine my remarks to that important stretch of the river, 22 miles long, which passes through the Administrative County of London.

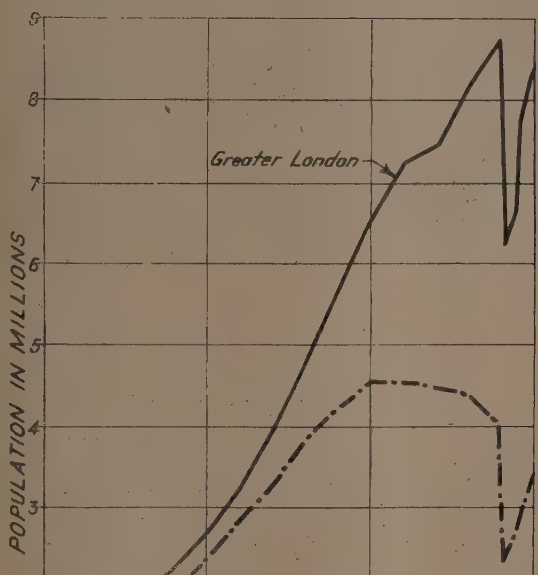


Fig. 1.—Population of London

Although the origin of the name London is Celtic, it is probable that there was no settlement of any importance on the site of London before the invasion of Britain by the Romans, under the Emperor Claudius, in A.D. 43. Geographically the site of London was ideal for the purpose of trade with the Continent and it was also far enough up the Thames for river crossings by ford or bridge to link up with the road system of the period. After the invasion, London rapidly grew in importance due to its position as a commercial centre and soon became the chief town in Britain.

The early town was built on two small hills between Blackfriars Bridge on the west and the Tower of London on the east and was divided by the Walbrook into approximately two equal areas. The site of the town provided a long frontage to the Thames and it is not improbable that the first engineering works to be constructed were wharves east of the Walbrook. The remains of substantial timber structures which have been discovered at various times, in the neighbourhood of Lower Thames Street, may well be the remains of these early works. About A.D. 150 London was surrounded by a protecting wall which enclosed an area of about 330 acres. The landward wall was about two miles in length and was of substantial construction. There was also a small unwalled settlement on the south side of the river. The foundations of the wall consisted generally of puddled clay and flints, although ragstone was sometimes used instead of flints. The main part of the wall was built of Kentish Ragstone with the internal and external faces roughly squared and built in courses, while the interior stones were laid in a rough state. The wall was built in a trench 3 or 4 ft. deep and about 10 ft. wide. The height of the original wall is not known, but during demolition work in Camomile Street in 1905, a length of wall 14 ft. 6 in. in height above the plinth was discovered. Remains of portions of the landward part of the wall have been discovered and some of these are still visible in various parts of the City. Unfortunately, little is known of the river wall which joined up with the landward wall and there appear to be no remains of it to-day.

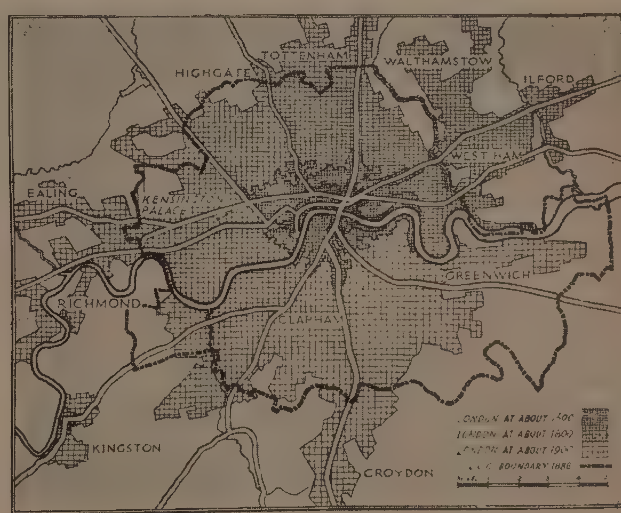


Fig. 2.—Growth of London, 1600-1900

*Given before a General Meeting of the Institution of Structural Engineers at 11 Upper Belgrave Street, London, S.W.1, on Thursday, October 9th, 1952.

London continued to prosper under the Romans, but after their withdrawal in A.D. 410 it declined and was probably only sparsely populated for some time. By the seventh century London appears to have become an important trading centre again, and is described by Bede, 673-735, as an emporium of many nations coming by land and sea. The growth of London continued throughout the centuries, and although efforts were made by several sovereigns to restrict its growth, these had little effect, and by 1801 the population of Greater London was just over 1,000,000. Since the beginning of the nineteenth century the population has increased at a phenomenal rate and in 1939 was 8,728,000. This rapid growth is shown by the graph plotted from the Census Reports of the Registrar General, Fig. 1. The growth of London during the period 1600-1900 is illustrated in Fig. 2, which also shows in a striking manner the expansion of London during the last century, when large areas of the countryside were absorbed for roads and buildings.

Although the river Thames was the main highway of London, it also constituted a barrier of no mean order which had to be surmounted. The river cut right across the commercial highways of the country in the early days of civilisation in this island, and means of crossing it had to be found. Apart from primitive boats, the first method of crossing a river was by fords and this was followed successively by ferries, bridges and tunnels. It is held by many that there was a ford near Westminster as the Roman highways appear to converge there and it is probable that there were others.

Ferries in course of time replaced the fords and were established at various positions along the Thames. One of the ferries, to which there are many historical references, was the horseferry which plied between Westminster and Lambeth. It is not known when it was first established but there is a specific reference to it in the year 1513. In that year the Archbishop of Canterbury granted to Humphrey Trevilyan the passage or ferry over the Thames from "Lamhithe" to the Bank of the Abbot and Convent of Westminster at a rent of 16 pence a year. It was provided, however, that the Archbishop, or any of his officers and servants, his goods

Fig. 3.—Old London Bridge, 1616
From an original print in the Guildhall Library

Fig. 4.—Old Putney Bridge

and chattels, should be transported free. When, in 1736, an Act was passed authorising the building of a bridge at Westminster, it provided for payment of compensation to the Archbishop and his lessees for damage to the horseferry and in 1750, when it was discontinued, a sum of £3,780 was paid.

The only ferry in the county in existence to-day is the free ferry between North and South Woolwich, which was opened in 1889, and conveys vehicles in addition to passengers.

Road Bridges

It is not definitely known when the first bridge was built over the Thames but the historian, Cassius Dio, writing nearly two centuries after the Roman invasion, describes the crossing of the river by the invaders, and he states that some of them forced the passage of the bridge. As the historian was writing nearly two centuries after the event, there must be considerable uncertainty whether a bridge did exist at that time. Although actual proof is lacking, it is almost certain that the Romans bridged the Thames during their occupation of Britain. It would be surprising if with their considerable experience of bridge building, they were satisfied with a ford or ferry to connect their road system, and it is known that Julius Caesar in the previous century crossed the Rhine by a bridge constructed by his engineers. In addition, there is evidence that the settlement of the land surface[1] in London amounts to about 15 feet in the last two thousand years indicating that the tide in Roman times did not extend so far up the river as it does to-day. With a shallower river and little or no tidal action the construction of a bridge would have been simplified.

The next historical reference to a bridge in London is in the second half of the tenth century when it was recorded that a woman, condemned to death for witchcraft, was drowned at London Bridge. From this time until the first stone bridge was built in the twelfth century there are many references to a timber bridge in London. The first stone bridge, now known as Old London Bridge, had 20 spans, and was in course of construction for 33 years from 1176-1209. It was built under the direction of Peter of Colechurch, Chaplain of St. Mary, Colechurch. He died in 1205 before its completion and was buried in the Chapel on the bridge.

Fig. 5.—Old Westminster Bridge
From a print in the L.C.C. Library

During 1581 and 1582 Peter Morice, a Dutchman established water works at the bridge for supplying the city with water. There were also water wheels for grinding corn at the Southwark side of the bridge.

According to the measurements made by Mr. Giles[2] in 1820, the length between abutments was 931 ft. and of

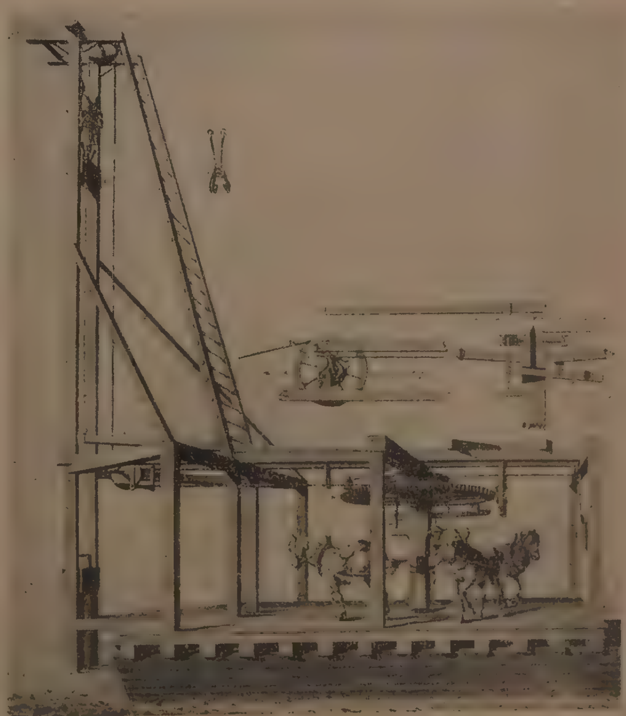


Fig. 6.—A perspective view of pile driver used at old Westminster Bridge
From a print in the L.C.C. Library

[1]ROYAL COMMISSION ON HISTORICAL MONUMENTS. Vol. III. Roman London.

[2]Min. Proc. Inst.C.E., Vol. II, 1843, p. 87.

this length, the piers occupied 406 ft. 10 in. and the aggregate water way between the piers above the starlings was 524 ft. 2 in., while below the starlings it was 230 ft. 11 in. The obstruction to the free flow of the water caused by the piers and starlings was so great that on the occurrence of a Spring tide and a high upland discharge, the difference at low tide in the level of the water above and below the bridge was about 5 ft. 7 in. The operation of shooting the bridge was very dangerous and many lives were lost through the upsetting of boats while passing under the arches. About 1760 all the buildings on the bridge were removed, the bridge was widened, and two arches were made into one, and in 1782 the bridge was freed from toll. The obstruction caused to the free flow of the river by the piers resulted in the water freezing in severe winters, and when this occurred it was the custom to hold fairs on the ice. The last one of any importance was held in 1814 and with the removal of the old bridge the freezing of the river ceased.

The bridge, which had been in existence for 622 years and had suffered from neglect, fire and flood, was demolished in 1831-34 for the sum of £35,500. It had played its part in sieges, rebellions, pageants and many historical events and during its existence London had grown into the most important city in the world. Citizens of London were very proud of their bridge ; Stowe in his "Survey of London," published in 1598 wrote, "To conclude of this bridge over the said River Thames, I affirm, as in other descriptions, that it is work very rare having with the drawbridge 20 arches of squared stone."

During the excavations for Adelaide House in 1921, an arch of Old London Bridge was discovered. It had a span of 29 ft. and a rise of about 7 ft. above the springing, and had been strengthened by the addition of three moulded ribs of Portland stone but unfortunately, this relic was not preserved.

For over five centuries, old London Bridge was the only bridge over the Thames and although London was extending westwards and growing rapidly in size and importance, the building of a second bridge was so vigorously opposed by the City of London that it was not until 1729 that a second bridge was built at Putney. On the occasion of the City of London advancing a loan of £100,000 to Charles II, the Minutes of the Common Council of 26th October, 1664, state: "And withal to represent unto His Majesty the City's great sense and apprehension of and most humble thanks for the great instance of His Majesty's good and favour towards them expressed in preventing of the new bridge proposed to be built over the River of Thames betwixt Lambeth and Westminster which as is conceived would have been of dangerous consequence to the state of the City."

Fig. 7.—Old Blackfriars Bridge
From an original print, 1790, in the Guildhall Library

A few years later in 1671 a Bill was presented to Parliament for authority to build a bridge between Putney and Fulham, but again, the opposition was so great that the Bill was rejected, and it was not until 1726, 55 years later, that an Act was passed authorising the building of Putney Bridge. This bridge was followed by the opening of old Westminster Bridge in 1750, old Blackfriars Bridge in 1769 and old Battersea Bridge in 1772. With the rapid growth of the population in the nineteenth century, the insistent demand for further means of communication across the river increased and during the century nine new road bridges were built, and the five earlier ones were all rebuilt. Since 1900, although bridges on new sites have been proposed, none has yet been built, but six have been rebuilt.

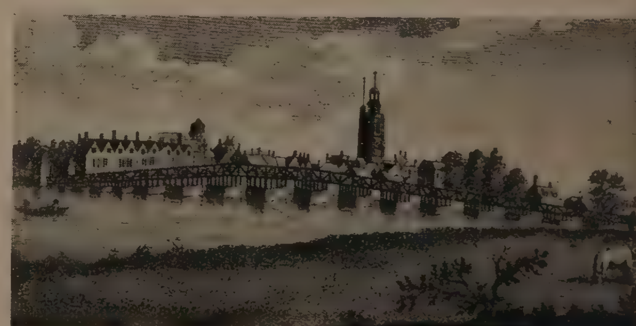


Fig. 8.—Old Battersea Bridge
From a print in the L.C.C. Library

All the Thames bridges, with the exception of London, Blackfriars, Westminster and Chelsea Bridges, built in the County before 1877, had been constructed by private companies for profit and tolls were payable for both vehicular and pedestrian traffic. It was found that people would travel considerable distances to avoid paying a small toll and consequently the enterprises were not generally financially successful. In 1877 the Metropolis

Toll Bridges Act was passed providing for the purchase by the London County Council's predecessors, the Metropolitan Board of Works, of all bridges, with the exception of Westminster, from Waterloo to Hammersmith inclusive, in order to open them to the public free of toll, and by the end of 1880 all the bridges had been freed. The amount of compensation paid by the Board to comply with the Act was £1,376,825. Blackfriars and Southwark Bridges were freed from toll in 1811 and 1864 respectively. Most of the bridges purchased had to be strengthened in order to carry the greatly increased traffic which resulted from the removals of the tolls, while Hammersmith, Putney and Battersea had to be rebuilt.

There are 14 road bridges over the River Thames in the County of London, ten are owned and maintained by the

Fig. 9.—Old Vauxhall Bridge

London County Council and the remaining four, Tower, London, Southwark and Blackfriars, by the Corporation of London. All of the original bridges, with the exception of Tower and Albert, have been rebuilt and I shall refer to each in chronological order and shall include one footbridge built by Brunel. It is interesting to notice the changes throughout the years in regard to the appearance, design and methods of construction of the bridges. Various particulars of the bridges are given in the appendix.

Old Putney Bridge, the first one to be built after old London Bridge, cost £23,000, and was opened in 1729. George II, when Prince of Wales, was reputed to be the first person to cross it. It was a timber bridge and originally had 26 spans, varying from 14 ft. to 32 ft., but in 1870 and 1871, in order to facilitate navigation, a number of the piers were removed and two larger spans were provided. A toll of 1s. was charged for every carriage with two horses, 6d. for a one horse carriage and ½d. for every foot passenger. The bridge was made free of toll in 1880 when it was found to be totally inadequate for the traffic which had increased about 300 per cent. on the abolition of the toll and it was therefore decided that the first new bridge across the river should be at this site. The old bridge, after a life of 157 years, was removed after the opening of New Putney Bridge in 1886. Its timbers had been so often renewed that it is doubtful whether any part of the original bridge above river bed level remained throughout its existence.

Old Westminster Bridge was built of masonry and had 15 semi-circular arches, the largest being 76 ft. The piers were constructed in heavy wooden caissons, which were built on the shore, floated into position and sunk on to the river bed, which had previously been prepared for them. It was built in the years 1739-1750, but before it was completed one of the piers failed and had to be rebuilt together with two adjoining arches. That the bridge took nearly 12 years to construct illustrates the difficulties of building a bridge of that size with the appliances then available. Fig. 6 shows a pile driver invented by Vanloue, a watchmaker, and used for the work. The increased scour of the river, attributed to the removal of old London Bridge, so weakened the foundations that the bridge had to be eventually rebuilt.

Old Blackfriars Bridge was a masonry bridge of nine elliptical arches, the largest having a span of 100 ft. It was supported on timber piles which were cut off at the required level by an invention by Mylne who designed the bridge. The piers were built in timber caissons similar to those used at old Westminster Bridge, but in this case, the bottoms of the caissons rested on the timber piles. The bridge with a gradient of 1 in 14 proved most unsatisfactory for horse traffic. In 1833 extensive repairs, costing £105,000, were carried out, but the increased scour of the river caused by the removal of old London Bridge made it unsafe and it had to be rebuilt.

Old Battersea Bridge was a timber structure opened in 1772 and replaced a ferry which had existed at the site for many years. When built, it had 19 spans, varying from 15 ft. 6 in. to 32 ft., but subsequently four of the spans were reconstructed to make two larger spans. After it had been purchased by the Metropolitan Board of Works in 1879 it was found that the decayed condition of the timber piles made it necessary to discontinue its use for vehicular traffic and powers were obtained to construct a new bridge. The old bridge, which cost only £20,000 to build, was removed in 1886 after being in use for over 100 years.

Fig. 10.—Old Waterloo Bridge

At the beginning of the nineteenth century there had been little building development in the neighbourhood of the site of old Vauxhall Bridge and the celebrated public resort known as Vauxhall Gardens was still in existence. It was at first proposed to build a stone bridge and Mr. John Rennie was appointed the engineer, but only a small amount of work was carried out before the proposal for a stone bridge was abandoned as it was decided to

economise by building a bridge with an iron superstructure. The new design was prepared by Mr. James Walker. This was the first iron bridge built across the Thames. It had nine spans of 78 ft. and each span had ten cast iron ribs of I Section, 18 in. deep. The ribs were cast in three sections, each section being cast with a spandrel framing above it in one piece. The joints of the ribs which were not machined were caulked with lead and covered with plates bolted to the ribs. Timber caissons were used for constructing the piers ; the site to be occupied by each was dredged to a depth of 3 ft. below the proposed foundation level and the excavation filled

Fig. 11.—Old Southwark Bridge
From an original print 1899, in the Guildhall Library

in with gravel and carefully levelled for the caisson to rest on. Subsequently, there was erosion at the middle arches, varying from 3 ft. to 6 ft., exposing the foundations of some of the piers, the increased scour being attributed to the removal of old London Bridge and old Westminster Bridge. To protect the foundations, iron ore and other heavy material was placed around the piers, but because of the continued risk to the foundations, the inadequate width of the bridge for traffic and the difficulty of handling vessels owing to the small span of the arches, it was finally decided to build a new bridge.

Parliamentary powers for building old Waterloo Bridge were obtained by a commercial company in 1809

Fig. 12.—Old Hammersmith Bridge
From a print in the L.C.C. Library

and the first design, which was prepared by Mr. George Dodd, was not adopted and subsequently Mr. John Rennie, who had already built a number of important bridges, was appointed engineer. The bridge, which was built of granite, had nine elliptical arches of 120 ft. clear span with piers 20 ft. thick. Rennie decided to use cofferdams for building the piers and abutments as he considered that caissons as then used would not be satisfactory. The cofferdams consisted of three rows of timber sheet piles driven about 15 ft. to 20 ft. into the ground ; the piles were tied together, well braced and strutted and the spaces between the piles were filled with puddled clay. The foundations of the bridge consisted of timber piles carrying a timber raft which was formed by laying transverse timbers on top of the piles with longitudinal timbers above. The spaces between the bottom timbers were filled with rubble and those between the top timbers with fitted blocks of stone. Over these was laid a 4 in. thick timber decking on which the bridge was built. The timber ribs forming the centering were built on a platform on the shore and floated into position between the piers on a specially prepared barge.

The work was started in 1811 and was first known as the Strand Bridge, but its name was changed to Waterloo Bridge by Act of Parliament in 1816 to commemorate "the brilliant and decisive victory achieved by His Majesty's Forces in conjunction with those of His Allies." The bridge was opened with great ceremony by the Prince Regent attended by the Duke of Wellington on June 18th, 1817.

In 1880 an investigation showed that the undersides of the masonry of the piers instead of being about 5 ft. below the river bed were from 1 ft. to 6 ft. above it and from 1882 to 1884 timber sheet piling was driven around each pier and concrete aprons provided at a cost of £63,000. In 1923 it was observed that part of the bridge

Fig. 13.—Old and New London Bridges
From an original print in the Guildhall Library

near the Lambeth side of the river had subsided and certain remedial measures were taken in the hope of arresting further settlement, but conditions so deteriorated that as a precautionary measure the bridge was temporarily closed to traffic for several months from May, 1924, when 1,400 tons of concrete and clay forming the base of the roadway, together with the parapets over the damaged arches, were removed and in addition timber supports were erected at arches Nos. 5 and 6, both of which were badly distorted owing to the relative settlements of the piers. At that time pier No. 5 had settled about 28 in. and the adjoining piers Nos. 4 and 6 had settled about 6 in. and 10 in. respectively. The spread of the footings was too large in proportion to the depth and considerable portions had broken away leaving a smaller area to carry the load, thus increasing the intensity of pressure on the foundations. In order that traffic might be maintained during the reconstruction of the bridge a temporary bridge, with a carriageway 20 ft. wide and two footways each 7 ft. wide, was built, being completed in August, 1925, at a cost of £164,000.

Various schemes were devised for dealing with Rennie's Bridge but after many years of fierce controversy the Council decided in 1934 to demolish the old bridge and

to build a new one with not more than five arches over the river and of a width sufficient to take six lines of vehicular traffic.

The demolition of the old bridge and certain preparatory works on the approaches cost £331,000.

Old Southwark Bridge, designed by Rennie, was a cast-iron arch bridge of three spans, the centre span being 240 ft. and each side span 210 ft. There were eight segmental ribs in each span, the depth varying from 6 ft. at the crown to 8 ft. at the springings. It was a massive structure and the weight of the ironwork was 5,780 tons, whereas a modern bridge of the same size would require only about 2,000 tons of high tensile steel. The bridge was supported on timber pile and raft foundations and the abutments and piers were built inside timber cofferdams similar to those used for old Waterloo Bridge. The bridge was removed to provide a wider one with improved gradients.

Fig. 14.—Hungerford Suspension Bridge
From a print in the L.C.C. Library

Fig. 15.—Old Chelsea Bridge

Old Hammersmith Bridge, opened in 1827, was the first suspension bridge built over the Thames in London. It had three spans, the centre being 400 ft. and the side spans about 145 ft. each. The bridge had a timber deck which was supported by cast iron cross girders. The uspension chains were carried by two towers of masonry, the same width as the bridge, but with an opening of only 14 ft. in each for both vehicular and pedestrian traffic. There were eight wrought iron chains, which were carried on cast iron saddles resting on rollers at the towers and these were anchored at the abutments. The bridge was too narrow and was considered unsafe for the increased traffic after it was freed from toll.

The present London Bridge, which followed the one built by Peter of Colechurch, is also of masonry. It is a substantial structure of five elliptical arches, varying in span from 152 ft. to 130 ft. and was the last Thames bridge designed by the elder Rennie, who did not however live long enough to prepare the detailed design. The completion of the design and the construction of the work was carried out by his son, Sir John Rennie. The bridge is supported on pile and raft foundations similar to those used for old Waterloo Bridge, but the footings of the piers are of more substantial construction. The cofferdams for the construction of the foundations were similar to those used for old Waterloo and old Southwark Bridges. Although the width of the bridge was considered ample for future requirements, it was later found that the footways were not wide enough for the large number of pedestrians who used it, and between 1902 and 1904 each footway was widened from 9 ft. 6 in. to 15 ft. by corbelling out the sides of the bridge.

Hungerford Suspension Bridge, opened in 1845, was designed and built by Isambard Kingdom Brunel for pedestrian use and had a width of 14 ft. The total length was 1,362 ft. with a centre span of 676 ft. and two side spans of 343 ft. each. It had two suspension chains, one above the other, on each side of the bridge, and the saddles on which the chains were carried rested on rollers working in oil.

It had a brief life as it was removed about 1860 to make way for Charing Cross Railway Bridge which incorporated a cantilevered footway. The chains and other iron work were removed to Clifton for the completion of the suspension bridge over the river Avon, the piers of which had been built by Brunel.

Old Chelsea Bridge was the property of the Government until it was purchased by the Metropolitan Board of Works under the Metropolis Toll Bridges Act of 1877. The bridge was built of wrought iron with the exception of the towers which were of cast iron. Originally there were two suspension chains on each side of the bridge, but although an additional chain was added later to each side, the load had to be limited to five tons per vehicle. The piers and abutments had timber pile foundations and were encased in cast iron in a somewhat similar manner to that of the piers of the present Westminster Bridge.

Westminster Bridge has seven spans, each with 15 arch ribs, which are of cast-iron except for a length at the crowns where they are of wrought iron riveted sections. The ribs were made in a number of sections and bolted together. The construction of the foundations is of considerable interest as the abutments and piers are supported on timber piles driven into the London clay and the piers are surrounded by cast-iron piles, 15 in. diameter, spaced about 5 ft. to 6 ft. apart, the spaces between being of ribbed cast-iron plates. Both piles and plates were driven into the clay. After the soft ground was removed from the area enclosed by the cast-iron, concrete was placed on the gravel which overlies the clay, to the top of the timber piles and the masonry built above. The foundations are further protected by a concrete apron extending from the clay to the top of the cast-iron piles. The abutments were built in a similar manner to the piers, but the cast-iron piles and plates were only used at the front and for a short length along each side. Westminster Bridge was owned by the Government until it was transferred to the Metropolitan Board of Works by the London Parks and Works Act in 1887.

Fig. 16.—Westminster Bridge

Old Lambeth Bridge, the superstructure of which was of wrought iron, was of the stiffened suspension type and had three equal spans, each of 268 ft. There were two groups of wrought iron cables on each side of the bridge, and these passed over towers at the piers and abutments to cast iron anchorage girders built in near the back of each abutment. The cross girders were riveted to two shallow longitudinal box girders, 2 ft. 3 in. deep by 1 ft. 6 in. wide, which were connected to the cables by vertical and diagonal members. Each river pier consisted of two cast iron cylinders, 12 ft. in diameter, constructed of segments bolted together internally, the lower parts being filled with concrete and the upper parts lined with brickwork 3 ft. thick. The cylinders were sunk about 18 ft. below the river bed into the London clay.

The bridge was opened in 1862, but 25 years later it was in such an unsatisfactory condition that Sir Benjamin Baker was asked to advise on the stability of the structure. He reported, "The design is such that the bridge cannot be considered a permanent structure, but with reasonable care it may be made to last another 30 years." He also recommended certain remedial works which were carried out and the maximum weight of any vehicle using the bridge was limited to three tons. The condition of the bridge may be surmised from the fact that two men were employed to watch the bridge and to close it if it seemed likely that a crowd would collect on it. In 1910 the bridge was closed to vehicular traffic, but was used by pedestrians until it was removed in 1929. The cost of the bridge, exclusive of approaches, was £35,000, and although it was economical in first cost it proved to be unsatisfactory in almost every other way.

Blackfriars Bridge, opened in 1869, is a five span arch structure and was of wrought iron. Originally it had nine ribs to each span but when the bridge was widened by 30 ft. in 1907-1909 to accommodate tramways, additional ribs of steel were added. The foundation for each pier was formed by sinking six wrought iron caissons in free air, and for the widening of the bridge a steel caisson was sunk under compressed air for each pier extension. The bridge is 105 ft. between parapets and is the widest over the Thames.

Albert Bridge is of an unusual type, and is unsatisfactory structurally. It has been described as a combination of the cantilever principle with suspension chains. The centre span is 384 ft. 9 in. and the two side spans are each 147 ft. 2 in. There are two towers at each pier, each tower being supported on a cast iron cylinder, sunk into the London clay and filled with concrete. Each cylinder is 21 ft. diameter at the base and diminishes to 15 ft. near the river bed level. The main supporting system consists of 32 ties which radiate from the tops of the towers and connect to the webs of the stiffening girders, the ends of which are anchored in cast iron pits, the radial ties being supported by vertical rods attached to the suspension chains and stiffening girders. The towers and portals are of cast iron, the stiffening and cross girders of wrought iron and the deck of timber. The bridge was opened in 1873, and 12 years later the wire ropes, having become dangerous, were replaced by chains of steel links. It has a five ton load limit and is scheduled for rebuilding within the next ten years.

Old Wandsworth Bridge, built of wrought iron, was of the continuous lattice girder type. The two main girders, 627 ft. long by 12 ft. deep, were supported on brick abutments and four piers, each of which consisted of two wrought iron cylinders, 7 ft. 6 in. diameter, filled with concrete. The cylinders were carried well down into the London clay and an enlarged base 13 ft. 6 in. in diameter was formed by excavating below the cylinders and filling with concrete. The load on the bridge was limited to five tons per vehicle.

Putney Bridge, built of granite, is the only masonry bridge that has been built since London Bridge. It has five segmental arches, the centre span being 144 ft. The method adopted for constructing the foundations of the piers is of interest, as a cofferdam of tongued and grooved timber piles was first driven around the site of each pier. After the water had been pumped out, three wrought iron caissons were built up within each cofferdam and sunk in free air to an average depth of about 24 ft. below the river bed into the London clay. Each caisson was 30 ft. by 26 ft. and had a double skin of wrought iron, with a space between them of 3 ft. 6 in. which was filled with concrete before sinking. The abutments were built in cofferdams of tongued and grooved timber piles. In order to restrict the waterway as little as possible nine wrought iron centres were used for supporting the arch stones of each span during construction.

Fig. 17.—Old Lambeth Bridge

In the years 1931-1933 the bridge was increased in width from 44 ft. to 74 ft. by widening on the downstream side. The extension of each pier was constructed on a steel caisson sunk under compressed air, while the extensions of the abutments were built in steel cofferdams. The whole of the granite facing was taken down and reused for the face of the widened portion. The arches for the widening were built on steel centres similar to those used for the construction of the original bridge, but the ends of the centres rested on sand jacks. An analysis of the stresses in the existing bridge was made by the elastic theory and it was found that the thickness of the arches could be reduced for the new work, for example, the thickness of the centre arch was reduced from 4 ft. 6 in. to 4 ft. at the crown and from 8 ft. 8 in. to 8 ft. at the springings. The reduction in thickness of the five arches saved about 12,000 cubic feet of granite.

Hammersmith Bridge is a suspension bridge with steel suspension chains supported on wrought iron towers encased in cast iron. The deck is of timber supported on wrought iron cross girders. The piers and abutments of the old bridge below road level were incorporated in the present structure, but the abutments were extended and the Surrey pier underpinned. It is only 19 ft. 9 in. wide at the towers and has a 15 ton load limit and is scheduled for rebuilding in ten to twenty years' time.

Battersea Bridge is a five span structure with seven segmental cast iron ribs, 4 ft. deep, in each span, each rib being cast in five sections and connected together by bolts. The spandrel framing, bracings and deck plates are all of wrought iron. The greater part of the footways are carried by cantilevers which project beyond the outer ribs ; these cantilevers are covered by ornamental cast iron covings. The piers and abutments were founded in the London clay and were constructed in timber cofferdams.

Fig. 18.—Blackfriars Bridge

Tower Bridge is the most easterly of all the bridges over the Thames, and although its architectural features

have received some criticism, I imagine it is to Londoners of to-day what Old London Bridge was to our forefathers. The bridge crosses the river at the world famous Pool of London and to avoid interference with sea-going shipping it was essential to provide an opening span. The bridge has three spans, the middle one is 200 ft. between the towers and is a double leaf bascule operated by hydraulic machinery. The average length of time that road traffic is interrupted by the opening of the bridge is about five minutes. The two shore spans, each 270 ft., are supported by suspension chains. Provision was made for pedestrians to cross above the opening span by two footways 141 ft. above high water level, but these are now closed to the public. The bridge is constructed of steel but the towers are encased with masonry. The two piers are larger than those of any bridge over the river Thames as they have to accommodate the rear ends of the opening leaves, etc. The foundation of each pier is 204 ft. 6 in. by 100 ft. and was constructed by sinking 12 open caissons well into the London clay. After these had been sunk to the required level the clay was excavated a further 7 ft. and undercut 5 ft. beyond the perimeter of the caissons. The arrangement and size of the caissons left a rectangular area, 124 ft. 6 in. by 34 ft. in the interior of the pier, which was not excavated and filled with concrete until the outer portion of the pier had been built to above high water level.

Vauxhall Bridge consists of five spans, the largest being 149 ft. 7 in. The piers and abutments were constructed in whole-tide timber cofferdams of tongued and grooved piles. Three of the piers were constructed without undue difficulty, but the cofferdam of the fourth pier gave very great trouble almost from the beginning, and a succession of mishaps culminated in its partial collapse which greatly delayed the commencement of the work on the superstructure. The superstructure is built of steel and each span has 13 two-pinned plated arch ribs. The ribs of the shore spans were erected on stagings, but as the other three spans had to be kept open for river traffic the three central ribs of these spans were successively erected, complete with bracings, on a pontoon and floated into position on the hinges prepared for them. The remaining ribs in these three spans were then erected in sections on timber beams attached to the bottom flanges of the ribs already in position.

Fig. 19.—Albert Bridge

Southwark Bridge is a steel bridge of five spans, each span having seven arched ribs. Half of each footway and the parapet are supported on cantilevers carried above the ribs. Each of the four piers is founded on a single steel caisson sunk under compressed air. The caissons had parallel sides and semicircular ends, the two larger being 102 ft. by 30 ft. Temporary caissons were built above the permanent ones to enable the piers to be built in the dry during sinking.

Lambeth Bridge has five spans, each with nine two-pinned steel arched ribs with segmental soffits. The seven inner ribs are spandrel braced at the haunches while the parts at the crown are fully plated. The two outer ribs are fully plated throughout their length. The ribs for the shore spans were erected on temporary timber stagings, but as the centre and two intermediate spans could not be closed completely to river traffic, the haunch sections of the ribs in these spans were built on timber stagings near the piers and the centre sections were brought to the site in a specially fitted barge, and lifted into position by cranes. After the ribs had been

Fig. 20.—Old Wandsworth Bridge

given their exact fabricated form and riveted up, the span of each rib was shortened by half an inch in order to reduce the possible tensile stress in the bottom flange. This made an allowance for any increase of stress should the abutments yield slightly under the horizontal thrust which amounted to over 3,000 tons. In the case of the ribs of the intermediate and centre spans the shortening was carried out by jacking from one end of the ribs, but for the shore spans the crowns of the ribs were jacked up from the river stagings and packings of appropriate thickness for the half inch shortening were inserted

Fig. 21.—Putney Bridge

between the hinges and the skewbacks. The abutments were constructed in cofferdams of steel sheet piles and timber frames. A single rectangular steel caisson, 108 ft. 1¾ in. by 37 ft. 0¼ in., was sunk under compressed air to an average depth of 27 ft. below the bed of the river to form the foundation of each pier. The temporary caissons were built of steel troughing in two tiers and the frames consisted of steel walings and timber struts. The weight of each caisson, etc., during sinking amounted to 3,100 tons.

Chelsea Bridge is a three span suspension bridge of the self-anchoring type. Unlike the old bridge it is almost free from ornamentation and is typical of the present day trend to obtain a satisfactory appearance without embellishments. The two suspension cables are each composed of 37 wire ropes of about 1⅞ in. diameter, grouped together in hexagonal shape. They are anchored to the ends of the stiffening girders and fixed to the tops of the towers. There are no portal bracings between the towers which are provided with hinged bearings at the bases, to allow for any necessary movement. The piers and abutments were constructed in steel sheet pile cofferdams with the upper frames of steel and the lower ones of reinforced concrete, the latter being incorporated in the foundations.

Wandsworth Bridge is of the deck cantilever type with three spans, the centre being 300 ft. and the shore spans 178 ft. 6 in. each between bearings. The shore spans form the anchor arms of the girders and the centre span consists of two cantilever arms of 90 ft. each and a suspended span of 120 ft. The bridge has seven main girders, the anchor and cantilever arms of the five inner girders being of lattice type, while those of the two outer girders and all the girders of the suspended span are of plate girder construction. High tensile steel was used for most of the superstructure. The abutments and piers were constructed in steel sheet pile cofferdams with one frame of reinforced concrete and the remainder of fabricated steelwork. The reinforced concrete frames were subsequently built into the concrete foundations. The anchor and cantilever arms of the girders were built out in both directions from the piers. The suspended span girders were each placed in position in one piece, the first two being lifted into position from a barge and the remainder lowered into position from the partially constructed deck.

Fig. 22.—Hammersmith Bridge

Waterloo Bridge is the only one of reinforced concrete over the Thames in the County of London. The proposal to demolish the old bridge undoubtedly caused great concern to the many admirers of Rennie's work, but the new one is a graceful successor to the one it replaced. It provides improved facilities for navigation and an adequate width for road traffic. It has five spans, part of the northern shore span being over the Victoria Embankment, and the new South Bank river wall has since been built to pass under the southern shore span. The girders and the tops of the piers are faced with Portland stone to harmonise with the buildings on the north bank. The superstructure is composed of two box

Fig. 23.—Battersea Bridge

girders joined by cross girders and the decking slab. The girders are continuous over two spans with cantilever ends in each half of the bridge and there is a suspended section in the centre span. The depth of the girders was kept as shallow as possible to provide the maximum headroom for shipping, and to obtain this result a high percentage of welded reinforcement was used. The superstructure is supported on reinforced concrete bearing walls, 2 ft. 3 in. thick, inside reinforced concrete shells which form the apparent piers and abutments, and protect the bearing walls from damage. The piers and abutments were constructed in steel sheet pile cofferdams with steel frames. The lower part of the sheet piling was left in the ground and in order to prevent relative settlement, reinforcement and dowel bars in the foundation block were welded to the piles.

Fig. 24.—Tower Bridge

Tunnels

Although tunnels have been constructed since the earliest times for dwellings, tombs, mining, water supplies, etc., their construction for the crossing of rivers is comparatively modern. The modern system of subaqueous tunnelling is undoubtedly due to the invention of the tunnel shield by Marc Isambard Brunel in 1818, and the use of compressed air working patented by Lord Cochrane in 1830. As early as 1798 there was a proposal for a tunnel under the River Thames at Gravesend, but this was abandoned. An attempt was next made in 1804 to construct a tunnel from Rotherhithe to Limehouse when a shaft was sunk and a heading over 1,000 ft. in length driven under the river; the difficulties encountered were so great that the work was suspended, and although continued later, it was finally deemed to be impracticable and abandoned. The first tunnel to be completed under the River Thames was constructed by Brunel between Rotherhithe and Wapping. The work began in 1825 but the tunnel was not opened until 1843; it was rectangular in section, 37 ft. 6 in. in width and 22 ft. 3 in. in height, and was divided into two sections by a central wall. It was for this work that Brunel invented the shield and used it for the first time. It differed from shields used later, as it was constructed of a number of separate sections, each of which could be moved forward independently of the others. There was considerable trouble from water which broke into the tunnel on several occasions and the work was suspended for lack of funds from 1828 until 1835 when the Government advanced money for the work to be continued with an improved shield. The tunnel was opened in 1843; financially it was a failure, and in 1865 was sold to the East London Railway Company and is still in use, forming part of the Underground Railway between Whitechapel and New Cross. The construction of the tunnel was an outstanding engineering achievement of the nineteenth century, and that it was completed successfully is a tribute to the skill and indomitable courage of its builder.

Fig. 25.—Vauxhall Bridge

The Tower Subway, constructed in 1869, was the next tunnel to be driven under the Thames. It was only 6 ft. 7 in. internal diameter and was constructed in the London clay by the aid of a circular shield which was forced forward by six screw jacks. The tunnel was intended for the conveyance of passengers by a carriage accommodating 12 people, drawn by a wire rope and

hauled by a steam engine of 4 h.p. in each shaft. The scheme was not a financial success and the tunnel was adapted for foot passengers, but was closed in 1899 and is now used to carry water mains.

In 1867 a tunnel[3] was proposed between Waterloo and Whitehall for a pneumatic railway and is of interest on account of some similarity to certain modern American

Fig. 26.—Southwark Bridge

practice. It was proposed to lay 10 ft. internal diameter tube in four lengths, each 235 ft. long, in a trench dredged in the river bed and it was intended that the end of each length should be supported on a pier formed by a cylinder, 21 ft. in diameter, sunk about 70 ft. below high water. The two piers nearest the south bank had been sunk and the bed of the river dredged for the first length of tube, but the work was discontinued owing to economic

Fig. 27.—Lambeth Bridge

conditions and it is believed that the cylinders were removed.

In the latter half of the last century the immense growth of the districts below London Bridge and the importance and extent of the businesses which had grown up on each side of the river, together with the growth of the Port of London, all drew attention to the urgent need for further facilities for cross river traffic in this area. As interference with river traffic had to be avoided, proposals were considered for tunnels, high level bridges and low level bridges with opening spans.

Fig. 28.—Chelsea Bridge

The demand for additional river crossings continued for some 20 years, and to meet this demand Woolwich Ferry was opened in 1889, Tower Bridge in 1894, Blackwall Tunnel in 1897, Greenwich Footway Tunnel in 1902, Rotherhithe Tunnel in 1908 and Woolwich Footway Tunnel in 1912.

BLACKWALL TUNNEL

The importance of this tunnel for cross river traffic is apparent when it is realised that the population to the East of London Bridge exceeded 1,500,000, and that prior to its construction there was no free crossing of the river between Tower Bridge and Woolwich Ferry, a distance of nearly nine miles.

The Act authorising the construction of the tunnel was passed in 1887 and the work began in 1892. The Metropolitan Board of Works had proposed three tunnels, two for vehicular traffic and one for foot passengers, and tenders for the footway tunnel had been obtained, but the London County Council, who replaced the Metropolitan Board of Works, decided that a vehicular tunnel should be built instead of the footway tunnel, and the latter was not commenced. Sir Alexander

[3] Min. Proc. Inst.C.E., Vol. CL, 1901-2, Part IV.

R. Binnie, at that time the Chief Engineer of the Council, who was responsible for the design and construction of the tunnel, records that great doubt was thrown upon the possibility of the scheme being successful, and it was held to be impossible by some of the leaders in the engineering profession, who no doubt had in mind the difficulties encountered by Brunel. Sir Benjamin Baker visited America on behalf of the Council to inspect the Hudson and Sarnia Tunnels which were then under construction. The external diameters of which were 20 ft. and 21 ft. respectively. He subsequently reported that he considered a tunnel to accommodate two lines of vehicular traffic could be constructed.

The tunnel, with approaches, is almost $1\frac{1}{4}$ miles in length, of which about one half, including the part under the river was constructed of cast iron with the aid of a shield and compressed air, the remainder consisting of cut and cover portions and open approaches. The tunnel is composed of cast iron segments, 2 ft. 6 in. wide, bolted together, filled with concrete and lined with glazed tiles. The external diameter of the tunnel is 27 ft., and accommodates a carriageway 16 ft. wide and two footways, each 3 ft. $1\frac{1}{2}$ in. There are two shafts of 58 ft. external diameter on each side of the river and the natural ventilation from these shafts was sufficient for many years for the cart traffic which then used it, but ventilating fans in the shafts have since been provided. The tunnel was driven within 5 ft. of the river bed for part of its length and as the intervening material consisted of coarse gravel it was decided to deposit from barges a layer of clay about 150 ft. wide to a maximum depth of 10 ft. in an endeavour to prevent the bed of the river from being blown up by the compressed air. On two occasions, however, holes were blown and water poured into the tunnel to a depth of about 8 ft., fortunately without injury to the men. The shield, which was specially designed for the work, was constructed of steel and weighed 250 tons and was driven forward by means of hydraulic rams which exerted a force of over 5,000 tons.

Fig. 29.—Wandsworth Bridge

It is interesting to note that there was no public supply of electricity available when the tunnel was completed so a generating station was built to supply the electricity at a cost of £30,000.

Fig. 30.—Waterloo Bridge

The whole of the work took a little over five years to complete, including one year for ancillary works, the under river portion being constructed in one year. The tunnel was the largest of its type at that time and the construction was undoubtedly carried out with great skill and resourcefulness. The cost of the work,

including £355,000 for property, was £1,320,000. It was opened to the public in 1897 and was at that time adequate for the cart traffic of the day, but the sharp bends at the shafts, the narrow roadway and the limited headroom made it totally inadequate for modern two-way traffic which amounts to over 10,000 vehicles a day.

Parliamentary powers were obtained in 1938 for the duplication of the tunnel and the designs were well advanced, but owing to the war and the conditions prevailing since, the work has been postponed, but is included in the first period, 1952-1961, of the County of London Development Plan, 1951. The proposed tunnel is designed for an external diameter of 30 ft. 6 in. with provision for a 20 ft. carriageway with 16 ft. minimum headroom. It will have no sharp bends, the smallest curve having a radius of 900 ft. To comply with the Act, it will be about 10 ft. lower than the old one, to allow the Port of London Authority to deepen the river if required in the future. When the new tunnel is completed it will be used for south-bound traffic and the old one for north-bound.

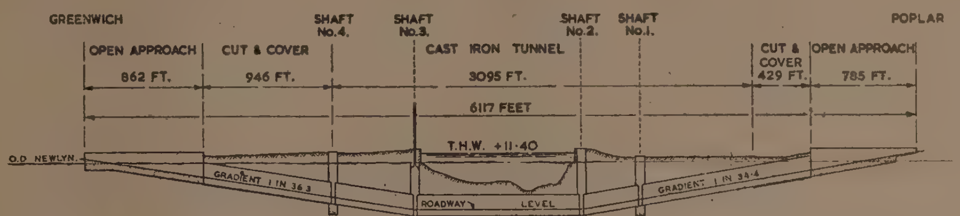


Fig. 31.—Blackwall Tunnel—Longitudinal Section

GREENWICH FOOTWAY TUNNEL

Ferries had for many years been in existence between the Isle of Dogs and Greenwich but the service was considered inadequate and in 1897 an Act was passed authorising the construction of a subway for pedestrians.

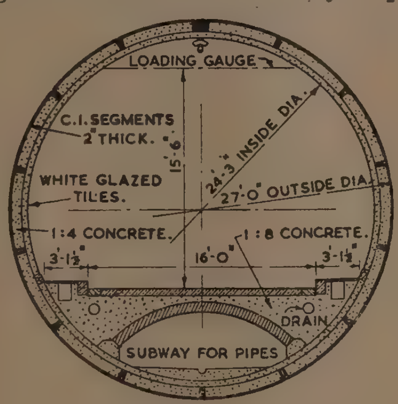


Fig. 32.—Blackwall Tunnel—Cross Section

The work consists of two shafts, one on each side of the river, connected by a cast iron tunnel 1,217 ft. in length, having an external diameter of 12 ft. 9 in. and providing a footway 8 ft. 8¾ in. wide. The tunnel was constructed by the aid of a shield and compressed air without undue difficulty. The shafts have an external diameter of 43 ft. and each contains a spiral staircase and an electric lift. The total cost of the work, including £58,000 for the acquisition of property and compensation to watermen, was £180,000. The work was carried out under the direction of Sir Alexander R. Binnie, at that time the Chief Engineer of the London County Council.

ROTHERHITHE TUNNEL

In 1892 it was proposed to provide a ferry near the site of the present tunnel, but owing to the opposition of the Thames Conservancy Board, this project was abandoned, and an Act authorising the construction of the tunnel was obtained in 1900 and work commenced in 1904. The total length is 6,883 ft. of which 2,020 ft. is in open approach, 1,122 ft. cut and cover and the remaining 3,740 ft. is tunnel lined with cast iron having an external diameter of 30 ft. and providing a carriageway 16 ft. wide with two footways each 4 ft. 8½ in. wide. The tunnel lining is similar to that used at Blackwall and there are also two shafts on each side of the river.

The tunnel was driven under compressed air, with the aid of two shields, each weighing 380 tons and the hydraulic rams could exert a force of 6,000 tons to force each shield forward. It is only about 7 ft. below the bed of the river, but the Thames Conservancy Board were unable in this case to allow clay to be placed on the river bed. The work was carried out with the utmost care and every precaution was taken to safeguard the men in event of the river bed being blown up. Fortunately there was clay above the tunnel and the river did not at any time break in. In order to obtain the fullest information about the strata, the contractors drove a pilot tunnel of 12 ft. 6 in. diameter across the river in

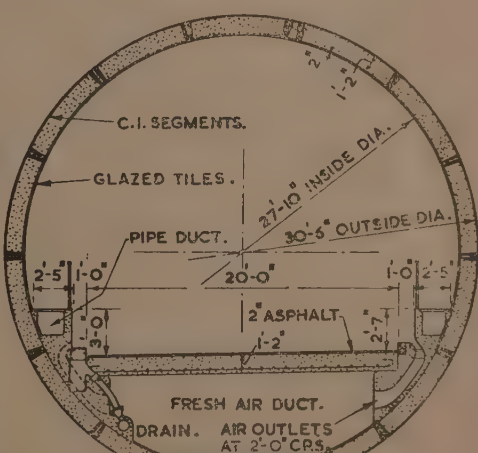


Fig. 33.—Proposed Blackwall Tunnel—Cross Section

advance of the main tunnel. This tunnel was lined with cast iron and the top was about 2 ft. below that of the main tunnel. No provision was made for artificial ventilation as at that time the natural ventilation was considered sufficient, but ventilating fans in the shafts have since been added. The contract time for the work was five and a half years but it was completed in just over four years, the under river portion being completed in nine months. The cost of the tunnel, including £517,000 for property, was £1,650,000.

WOOLWICH FOOTWAY TUNNEL

On many occasions the ferry service had to be suspended owing to fog, causing great inconvenience to the

public, and to supplement it, a footway tunnel was constructed under the river, Parliamentary authority being obtained in 1909. The design of the tunnel closely resembles that of Greenwich Footway Tunnel, but is 1,655 ft. long. The cost of the work was £86,000, the cost of property, etc., being only £550.

Rotherhithe Tunnel and Woolwich Footway Tunnel were carried out under the supervision of Sir Maurice Fitzmaurice, at that time the Chief Engineer of the London County Council.

Embankments

The River Thames passed through many low-lying areas which are liable to be flooded at high water in the event of the failure of the embankments. Within the County of London there are about ten square miles below Trinity High Water mark and about 22 square miles below a level of 16 ft. 9 in. above Ordance Datum (Newlyn).

There are records extending over many centuries of the Thames overflowing and causing great hardship and loss of property. The highest tide recorded occurred on 6/7th January, 1928, and exceeded the previous highest tide, recorded in 1881, by as much as 11 in. The actual height of the tide at London Bridge in 1928 was 5 ft. 10 in. above the predicted height and serious flooding occurred at many places, unfortunately with loss of life. Exceptionally high tides are caused by surges set up or propagated into the North Sea, but the height of the tide in the County of London is dependent upon the magnitude of the surge arriving at Southend and its relation to the time of high water of a spring tide and the volume of the upland discharge. It is of interest to recall that in 1717 the river fell so low that people were able to walk across it.

The London County Council under the Thames River (Prevention of Floods) Acts, 1879-1929 are responsible for exercising general supervision of all river banks within the County, but it is the duty of the riparian owners to provide and maintain the flood defence works.

As a result of the flooding in 1928 a conference was summoned at the instance of the Prime Minister to review the situation and in March, 1930, the Council decided that the flood protection works within the County should be raised to a level of 17.00 ft. above Ordance Datum (Newlyn) at the eastern boundary of the County rising by steps to 18.17 ft. above Ordance Datum (Newlyn) at the western boundary.

A system of flood warnings has been devised whereby the public are notified by the police when the tide reaches a danger level which would affect a prescribed district. Automatic alarm apparatus has been installed at a number of places including Southend and Erith and gives warning in adequate time for the necessary action to be taken within the County.

Some of the embankments which now confine the tidal portion of the Thames and protect the low lying areas are of ancient date and we can only surmise on the period in which they were built. The forms of construction were many and varied. Many attempts have been made in the past to improve the banks of the river within the County and Sir Christopher Wren proposed an embankment on the north side in connection with his plans for rebuilding London after the great fire in 1666, and an Act was passed to prevent any building within forty feet of the river from London Bridge to the Temple. This open space was afterwards known as the Forty-Foot Way, but numerous encroachments were made from time to time and the Act was repealed in 1821.

In the last century many proposals for embanking both sides of the river and reclaiming the large areas occupied by the mud banks were made, and many detached lengths of embankment wall were built, but of those built during the last century I shall only refer to the three important lengths of wall built by Sir Joseph Bazalgette between 1864 and 1874. The combined lengths of these embankments exceed $2\frac{3}{4}$ miles and the building of these, together with ancillary works, constituted one of the most important improvements of London and its river and have added to the dignity of the capital and to the pleasure of its people.

Victoria Embankment

In 1847 it was made compulsory to drain houses into sewers and these discharged their contents into the Thames at about low water level. At the wider parts of the river there were mud banks of large extent and these became so polluted with sewage that the state of the river was a cause of great anxiety and in 1856 an Act was passed authorising the construction of intercepting sewers for preventing the sewage going into the Thames near the metropolis. The construction of one of these sewers under the Strand would have caused great inconvenience to traffic and eonsideration was given to laying the sewer along the foreshore of the river.

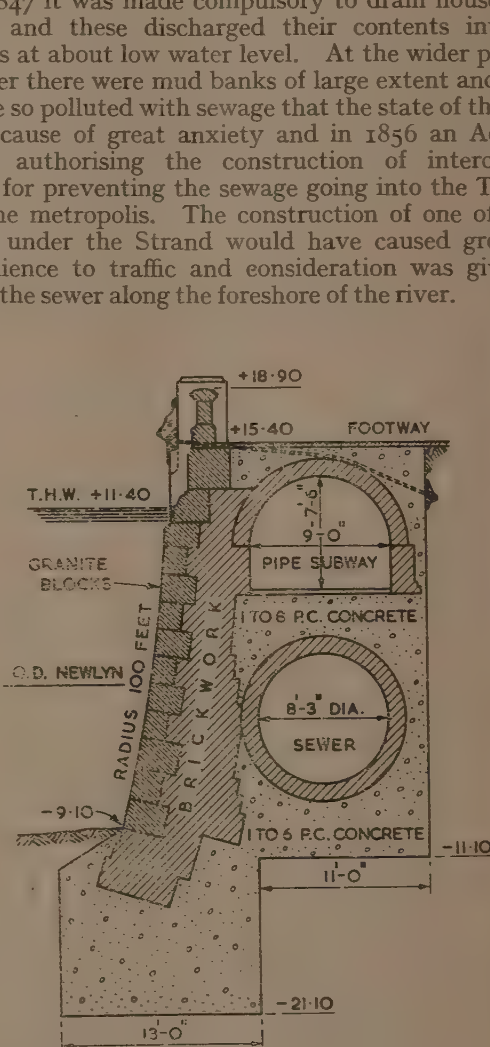


Fig. 34.—Victoria Embankment Wall

A Select Committee, appointed in 1860 to consider the best means of providing for the increasing traffic of the metropolis by the embankment of the Thames reported, "The embankment of the north side of the Thames from Westminster Bridge, to or nearly to Southwark Bridge, would afford a desirable mode of improving the banks and the bed of the river and facilitate the construction of the low level sewer along the foreshore; while a roadway on the embankment would greatly relieve the crowded thoroughfares."

The Parliamentary Powers obtained in 1862 authorised the Metropolitan Board of Works to construct a solid embankment and roadway 100 ft. in width from Westminster Bridge to Temple Gardens, the remainder of the roadway to Blackfriars Bridge to be 70 ft. in width and carried on arches to allow the river to flow to the

adjoining wharves. Subsequently, however, in connection with the construction of the Metropolitan District Railway, powers were obtained to construct the solid embankment with a roadway 100 ft. in width for the whole length of the work. The embankment extends from Westminster Bridge to Blackfriars Bridge and is about $1\frac{1}{4}$ miles in length. The area of land reclaimed from the river was $37\frac{1}{4}$ acres of which 19 acres were used for the roadway and eight acres for public gardens. The reduction in the width of the river was considerable and near the present Charing Cross Bridge it was reduced from 1,480 ft. to about 1,040 ft. The wall, founded about 14 ft. below the bed of the river, incorporates the Northern Low Level Sewer and a subway for gas, water, etc., and is constructed of brickwork and concrete faced with granite. Whole tide dams or cofferdams were used throughout the work and were carried down into the London clay. For the greater part of the length extending from Westminster Bridge to just beyond Waterloo Bridge a novel form of construction was adopted for the dams, which were formed of oval wrought iron caissons, 12 ft. 6 in. long by 7 ft. wide with a cast iron cutting edge. The caissons were sunk close together in the front of the wall, with or without the use of compressed air, with their longer axes parallel to the river. Watertight joints were provided at the junctions of the caissons, the lower parts of which were filled with concrete, and incorporated in the toe of the wall. Downstream of Waterloo Bridge the greater part of the wall was built behind dams consisting of two rows of timber piles, spaced 6 ft. apart, which were driven into the London clay and extended above high water level. The ground between them was excavated down to the clay and the space to the top of the piles was then filled in with puddled clay. The remainder of the wall was built in cofferdams, the sides of which were of similar construction to that of the dams.

The work was divided into three contracts and was started in 1864 and opened to the public in 1870; a remarkably short time when the extent of the work, the plant available and the types of dam used, are considered. The gross cost of the embankment and ancillary work, including property was £1,552,000.

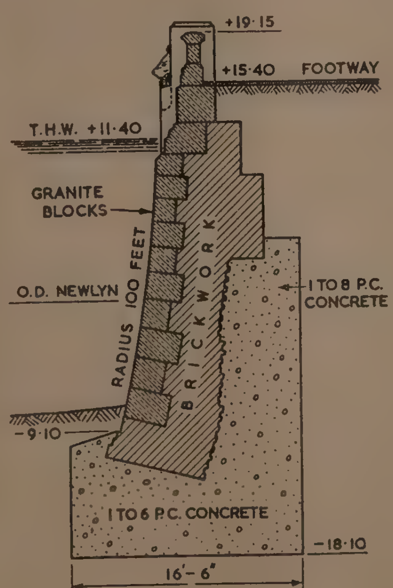


Fig. 35.—Albert Embankment river Wall

ALBERT EMBANKMENT

The Albert Embankment, which was completed in 1868, is on the South Side of the river between Westminster Bridge and Vauxhall. It is 4,300 ft. in length and during its construction $9\frac{1}{2}$ acres of land were reclaimed from the river, although, upstream of Lambeth Bridge, part of the river was increased 120 ft. in width. A 20 ft. promenade and part of St. Thomas's Hospital were built on the reclaimed land. The wall, constructed of concrete and brickwork faced with granite, was built in a whole tide timber cofferdam for over half of its length. Of the remainder, part was constructed behind a dam consisting of a single row of timber piles, and where the river was widened, the wall was built in trench. The work took about $2\frac{3}{4}$ years to complete and the gross cost was £1,154,000.

CHELSEA EMBANKMENT

This embankment, completed in 1874, is situated on the north side of the river between Chelsea Bridge and Battersea Bridge and is 4,130 ft. in length, and, as in the case of the Victoria Embankment, its construction facilitated the laying of the Low Level Sewer. There was not a suitable road near the river in which the sewer could be laid and as the cost of constructing the sewer under the foreshore behind a dam would have been expensive, it was decided to build an embankment and a roadway in which the sewer could be laid. The wall which is constructed of concrete faced with granite was built in a half tide timber dam. A much smaller cross section was used for this wall as there was a good foundation a few feet below the bed of the river. The land reclaimed from the river was $9\frac{1}{2}$ acres in extent and was used chiefly for the construction of a roadway 70 ft. wide. The gross cost of the work, which took nearly three years to complete, was £349,000.

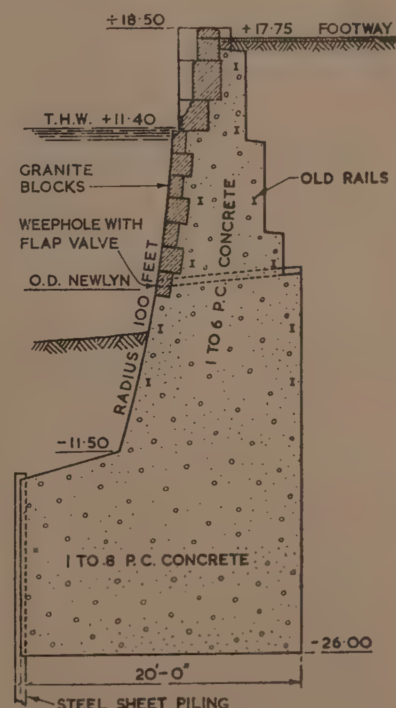


Fig. 36.—South Bank River Wall

The South Bank of the River Thames

COUNTY HALL RIVER WALL

For many years there was a marked contrast between the north and south banks of the river between Westminster and Blackfriars Bridges, and the construction of the Victoria Embankment and the imposing buildings on

the north side accentuated the mean appearance of the south. The London County Council had for many years wished to improve the south bank and it was partly for this reason that County Hall was built there. In order to provide a suitable frontage to the building a new river wall was built as a continuation of Albert Embankment and 2½ acres of land was reclaimed from the river. The wall, 1,015 ft. long, is of concrete faced with granite and was built in three sections, the first being completed in 1910 and the last in 1931. It is founded on London clay and the greater part was built behind a dam consisting of a row of tongued and grooved piles, but the last section was constructed in a steel sheet pile cofferdam. The foundation of the wall is 6 ft. lower than the south abutment of Westminster Bridge, which is built on timber piles and to avoid damage to it a row of steel sheet piling was first driven to protect the abutment foundations and a steel caisson was sunk under compressed air to form the foundation of the first 47 ft. of the wall. The cost of the wall was £117,000. The first and larger section of the work was carried out under the supervision of Sir Maurice Fitzmaurice, then the Chief Engineer of the London County Council.

During the excavation for the foundations of County Hall, the remains of a Roman boat were discovered together with coins of the first and third centuries and various British and Roman objects. The boat is now in the London Museum.

South Bank River Wall

The London County Council had obtained Parliamentary powers in 1939 to extend the river wall to Waterloo Bridge and to acquire lands for the improvement of the area, but owing to the war and the conditions prevailing afterwards, it was not until 1948 that it was possible to proceed with the work in connection with the proposal to hold the Central Exhibition of the Festival of Britain at the South Bank. In the meantime, the appearance of the area had deteriorated further owing to bomb damage and the storing of material from demolished air raid shelters and buildings.

The wall, which extends from County Hall to Waterloo Bridge, is about 1,700 ft. in length and during its construction about 4½ acres of land were reclaimed. Under the river deposits there was a bed of ballast overlying the London clay, the ballast varying in thickness from 5 ft. near County Hall to 20 ft. by Waterloo Bridge. The first 360 ft. of the wall was founded on clay, about 20 ft. below the river bed, at a level of – 26.00 N.D. but the remainder was founded on the ballast at levels varying from – 12.00 N.D. to – 18.00 N.D. The wall consists of mass concrete faced with granite and was built in whole tide cofferdams, formed of steel sheet piling with steel walings and timber struts. The steel piles were driven well into the London clay and when the cofferdams were removed the front piles were cut at the toe of the wall and the lower portion left in the ground. Many of the old river walls have cracked as the result of thermal movements and in order to reduce cracking, keyed expansion joints were formed 180 ft. apart and old rails were incorporated in the wall. The work was commenced at the end of the County Hall river wall and was carried out in short sections. The material from the demolished air raid shelters, etc., was used for filling behind the wall and was a most satisfactory material for the purpose, and bulldozers were used to place and compact it in thin layers.

The work, which took 21 months to carry out, was completed in 1950 at a cost of about £400,000. The works were carried out under the supervision of the Chief Engineer of the London County Council, Mr. J. Rawlinson, M.Eng., M.I.C.E., and Mr. Robert H. Matthew, A.R.I.B.A., advised on the architectural treatment.

I cannot conclude my address without expressing my admiration for all those concerned with these works, the engineers, the craftsmen and workmen. Many of the works were carried out under dangerous and difficult conditions, but by skill, courage and perseverance they were completed and have contributed to the greatness of London.

Acknowledgements

I desire to express my thanks to the London County Council for permission to include information from records and photographs and to the Chief Engineer, Mr. J. Rawlinson, M.Eng., M.I.C.E., to include works under his control, and to the officers of the Corporation of London for information regarding City bridges, etc. Fig. 2 by permission of the London Museum.

APPENDIX

Bridge	Date of Opening	Engineer	Architect	Type of Bridge	Number of Spans	Clear Spans	Length between Abutments	Width between Parapets
London Several Bridges of Timber	Date of first not definitely known							
Old London	1209	Peter of Colechurch		Masonry Arch	20		931 ft.	
Old Putney	1729	Sir Jacob Ackworth		Timber	26	14 ft. to 32 ft.	764 ft.	22 ft. 6 in.
Old Westminster	1750	Mr. Charles Labelye		Masonry Arch	15	25 ft. to 76 ft.	1220 ft.	44 ft.
Old Blackfriars	1769	Mr. Robert Mylne		Masonry Arch	9	70 ft. to 100 ft.	935 ft.	42 ft.
Old Battersea	1772	Mr. Holland		Timber	19	15 ft. 6 in. to 32 ft.	734 ft.	24 ft.
Old Vauxhall	1816	Mr. James Walker		Cast Iron Arch	9	78 ft.	809 ft.	36 ft. 3 in.

APPENDIX—*continued*

Bridge	Date of Opening	Engineer	Architect	Type of Bridge	Number of Spans	Clear Spans	Length between Abutments	Width between Parapets
Old Waterloo	1817	Mr. John Rennie		Masonry Arch	9	120 ft.	1240 ft.	42 ft. 6 in.
Old Southwark	1819	Mr. John Rennie		Cast Iron Arch	3	210 ft. and 240 ft.	708 ft.	42 ft. 6 in.
Old Hammersmith	1827	Mr. W. Tierney Clarke		Wrought Iron Suspension	3	400 ft., 145 ft. 6 in. and 142 ft. 6 in.	734 ft.	25 ft. 10 in
London	1831	Mr. John Rennie		Masonry Arch	5	130 ft. to 152 ft.	782 ft.	65 ft.
Old Chelsea	1858	Mr. Thomas Page		Wrought and Cast Iron Suspension	3	333 ft. and 165 ft.	703 ft. 2 in.	47 ft. 3 in.
Westminster	1862	Mr. Thomas Page		Wrought Iron and Cast Iron Arch	7	94 ft. 4 in. to 120 ft.	810 ft. 7 in.	84 ft. 3½ in
Old Lambeth	1862	Mr. P. W. Barlow		Wrought Iron Stiffened Suspension	3	268 ft.	828 ft.	31 ft. 9 in.
Blackfriars	1869	Sir Joseph Cubitt		Wrought Iron and Steel Arch	5	155 ft. to 185 ft.	922 ft. 6 in.	75 ft. Widened in 1909 to 105 ft.
Albert	1873	Mr. R. M. Ordish		Wrought Iron Suspension on the Ordish-Lefeuvre System	3	384 ft. 9 in. and 147 ft. 2 in.	709 ft. 1 in.	41 ft. 4½ in
Old Wandsworth	1873	Mr. R. M. Ordish		Wrought Iron Continuous Lattice Girder	5	106 ft. and 126 ft.	620 ft.	29 ft. 9 in.
Putney	1886	Sir Joseph Bazelgette		Masonry Arch	5	112 ft. to 144 ft.	700 ft.	44 ft. Widened in 1933 to 74 ft.
Hammersmith	1887	Sir Joseph Bazelgette		Wrought Iron and Steel Suspension	3	400 ft. 6 in., 146 ft. 0 in. and 143 ft. 6 in.	734 ft.	42 ft. 10 in
Battersea	1890	Sir Joseph Bazelgette		Cast Iron Arch	5	113 ft. 3 in. to 163 ft. 2 in.	725 ft. 6 in.	40 ft. 3 in.
Tower	1894	Sir John Wolfe-Barry	Sir Horace Jones	Steel Two Leaf Bascule and Suspended Side Spans	3	Opening 200 ft. Side 270 ft.	880 ft.	Opening 50 ft. Side 60 ft.
Vauxhall	1906	Sir Maurice Fitzmaurice	Mr. W. E. Riley	Steel Arch	5	130 ft. 6 in. to 149 ft. 7 in.	760 ft.	80 ft.
Southwark	1922	Sir Basil Mott	Sir Ernest George, R.A.	Steel Arch	5	126 ft. to 136 ft.	708 ft. 6 in.	55 ft.
Lambeth	1932	Sir George Humphreys	Sir Reginald Blomfield, R.A.	Steel Arch	5	125 ft. 2 in. 165 ft.	776 ft.	60 ft.
Chelsea	1937	Messrs. Rendel, Palmer and Tritton	Messrs. G. Topham Forrest and E. P. Wheeler	Steel Self Anchored Suspension	3	332 ft. and 163 ft.	698 ft.	82 ft.
Wandsworth	1940	Sir T. Peirson Frank	Messrs. E. P. Wheeler and F. R. Hiorns	Steel Cantilever	3	165 ft. and 284 ft.	646 ft.	60 ft. 6 in.
Waterloo	Partial use 1942 Complete 1944	Messrs. Rendel, Palmer and Tritton in association with Sir T. Peirson Frank	Sir Giles Gilbert Scott O.M., R.A.	Reinforced Concrete Continuous Girders with Cantilevers and Suspended Span	5	232.2 ft. and 238.7 ft.	1236 ft.	79 ft. 9 in.

Prestressed Concrete Bridges and other Structures*

By Donovan H. Lee, B.Sc.(Eng.), M.I.C.E., M.I.Mech.E., M.I.Struct.E.(Member of Council)

The structures described in this paper are in the main prestressed on a system utilising high tensile alloy steel bars with specially threaded ends and nuts.

The system was described in the technical press ([1],[2],[3],[4]) at the time manufacture of the steel and special components commenced. Descriptions have been given previously of various tests, in particular those carried out in 1949-50 by the Field Test Unit of the Ministry of Works. These tests were referred to by Mr. O. J. Masterman in a paper[5] to this Institution in October 1950, and were subsequently described in detail by Professor A. D. Ross.[6] As some of the earlier applications were mentioned fairly widely in the technical press[7],[8],[9],[10] no repetition of these will be made.

In the last two years or so there have been many applications of the use of this high tensile alloy steel for pre-stressing and some for other purposes, so for brevity it will be desirable to restrict mention in this paper to some typical and a few special applications of technical interest. These examples are grouped roughly in the following order:—buildings, miscellaneous structures, strengthening existing structures, and new bridges.

Minimum Economic Length for Post-tensioning

In common with other systems of post-tensioning there is a lower limit of length of member below which the cost of the end anchorage and the stressing operation, being virtually constant, makes it uncompetitive compared with other materials of construction. Generally, below this lower limit, which varies between 15 and 25 feet, ordinary reinforced concrete, either pre-cast or poured in place, becomes cheaper than post-tensioned prestressed concrete. As in the case of beams the latter may have only half the weight of concrete for equal carrying capacity, and in the case of hollow poles about a third, transport costs make the minimum competitive length difficult to indicate more than very roughly. Pre-tensioned concrete, precast in a factory specialising in this class of work also becomes cheaper than post-tensioned concrete when the length of member becomes less than a certain figure which varies, according to the delivery distance particularly, and to the other usual factors affecting cost, so that it is usually cheapest, provided there is enough repetition, for all short members.

Fig. 1.—Stressing of roof valley beams for B.C.U.R.A. building at Leatherhead

The post-tensioned concrete, however, has a clear almost undisputed field of lowest cost for an ever-increasing variety of concrete members subject to bending where the length of the member exceeds the greater of the rough length limits just mentioned. Where the shear is great or the span is great, post-tensioned concrete has a definite cost advantage. Notwithstanding that generally if there is a large amount of repetition of short members, pre-tensioned units made by a "long-line" method will normally be cheaper, post-tensioned concrete members of even quite short length can be competitive in certain applications.

As an example of the ability of post-tensioned concrete to compete in factory-made members of short length with a fair amount of repetition might be cited

**Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1. on Thursday, 11th December, 1952 at 6 p.m.*

ıollow tubular concrete members post-tensioned with a entral bar and used as lighting standards and transnission poles. Of shorter length, production of post-ensioned concrete railway sleepers in Germany might ›e mentioned.

Buildings

Although post-tensioned concrete members are made o a large extent in factories making other pre-cast con-rete products and no expensive plant or equipment is ıeeded for this, it is natural that in many cases where uitable aggregates are available locally that the pre-tressed concrete should be made at the site. Making up nembers at the site from factory made units is an alter-

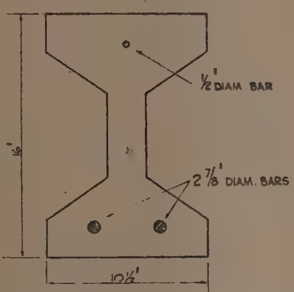


Fig. 2.—Section of roof beams in extension to the works of the Bristol Stone and Concrete Company

native. Apart from the forms, the sides of which are usually re-used almost daily, and some portable stressing equipment, no special plant is needed. Fig. 1 shows pre-stressed roof beams being stressed preparatory to being sent to the site and hoisted into position. The author is indebted to Messrs. C. W. Glover & Partners, the Consulting Engineers responsible for this work, for this illustration. Fig. 11 shows another example of beams made at a pre-cast concrete factory. These beams are a typical example of comparatively small pre-stressed beams made in sections and subsequently jointed and stressed. The section of the beams is shown in Fig. 2, the clear span being 25 ft. and the design load

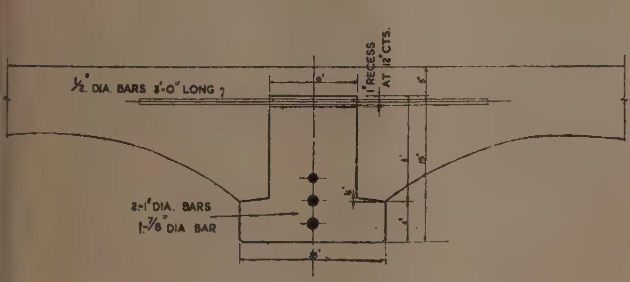


Fig. 3.—Prestressed roof beams

(including a dead weight of 1.4 tons) was 9.5 tons. Beams of a similar size are shown in Fig. 3 and were made at a precast concrete works in Hertfordshire for delivery to Coventry. As an example of small beams Fig. 4 shows a number being stressed by the London Co-operative Society, the length being 20 ft.

Fig. 5 shows prestressed 3-hinged frame members forming the skeleton framework of a new church in the Bristol area. There is a hinge at the apex and the arrangement of the bars is shown in Fig. 6.

This is a case, like portal frames, where site stressing is used to connect together pre-cast units, which in some cases are already pre-stressed.

While post-tensioning of short members is not generally competitive it is sometimes economical because it enables parts of frames to be stressed together at the site; it also allows members made up of a number of blocks

Fig. 4.—Small roof beams made and stressed by the London Co-operative Society

to be assembled and stressed at the site. In the case of the reconstruction of Yarmouth South Town Station by British Railways, Eastern Region[14] an interesting combination of different methods is being used. Fig. 7 shows a cross-section and for the single cantilever members the

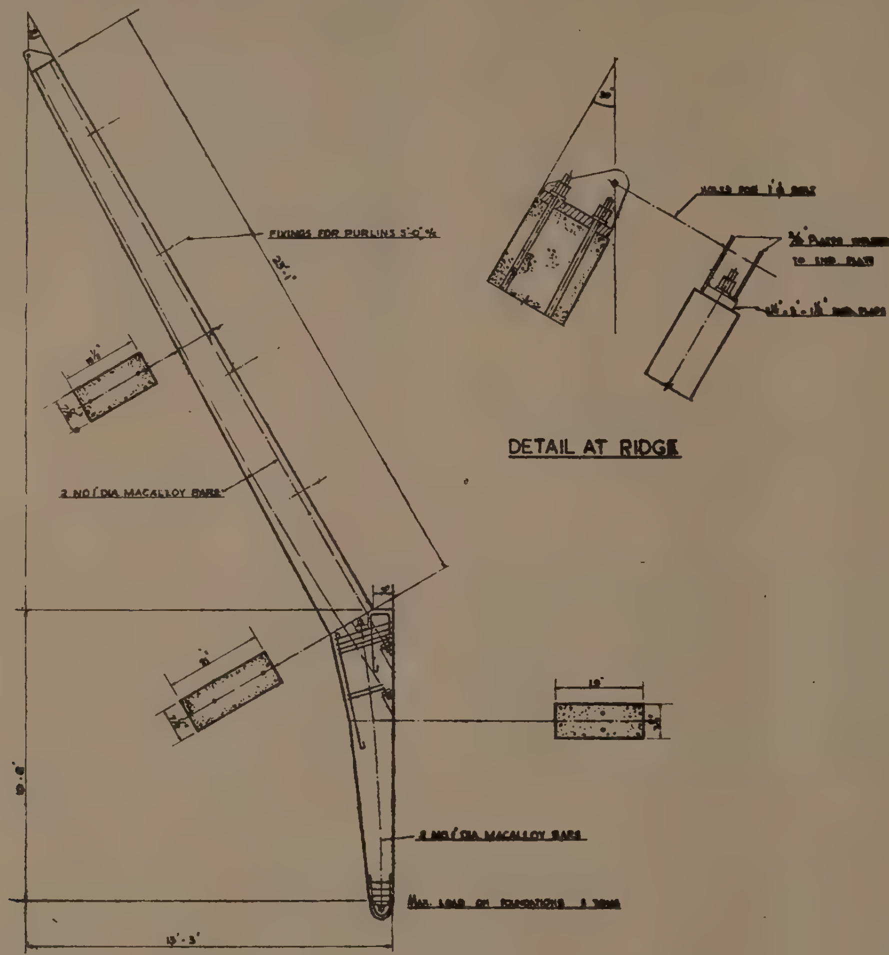
Fig. 5.—Prestressed 3-hinged frames in new church at Bristol

Freyssinet system is to be used, while the remaining precast units are to be pre-tensioned and also factory made. The final assembly and stressing at the site is being done with the high tensile steel bars now being discussed.

Long-span Floors

While for precast concrete floors ordinary hollow reinforced concrete is generally more economical than pre-

stressed concrete below about 15 ft. span, the weight of precast concrete floors increases for greater spans and pre-tensioned concrete is being increasingly used for greater spans. Post-tensioned precast concrete floors are also competitive for spans over 20 ft. and for considerably greater spans give an opportunity not previously existing to have large spans either poured in place or precast without excessive dead weight and high cost. Theoretically, that is to say considering the stresses alone, it is possible to have post-tensioned floors of spans up to 40 ft. with a depth not exceeding 10 in., assuming a hollow construction as either Fig. 8a or 8b.



Detail of Frame

Fig. 6.—Detail of 3-hinged prestressed frames shown in Fig. 5

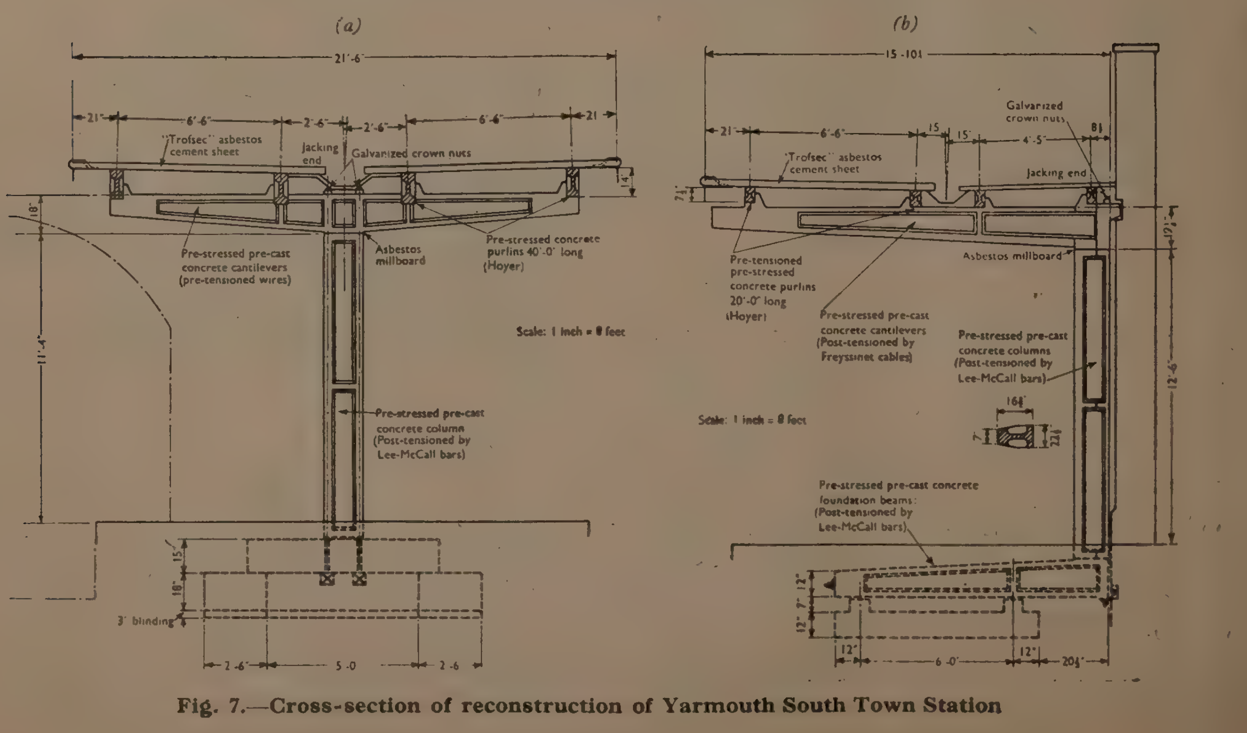


Fig. 7.—Cross-section of reconstruction of Yarmouth South Town Station

However, in the author's opinion it is desirable to limit these spans so as not to exceed a span to depth ratio of 30, except in cases where it is known to be impossible for there to be live load applied and reapplied rhythmically so as progressively to increase the elastic deflections with each application of the load. With further experience of the natural frequencies and natural damping of longer span floors of this type it is possible that their use will increase noticeably in the near future. In the case of flat roofs, these precautions are not so important, but with both large span floors and roofs it is more economical in steel and generally also in cost to use a type of construction that is light (for example a ribbed

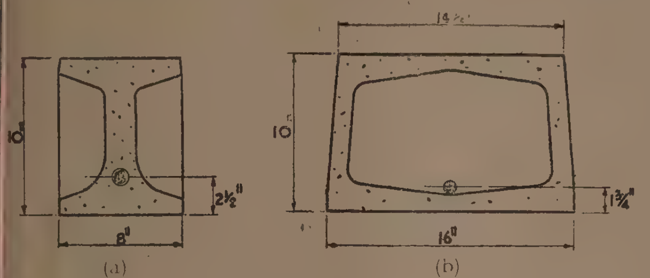


Fig. 8.—Alternative types of post-tensioned long span floors and roofs

type of floor) but not over shallow in constructional depth.

As an alternative to the use of precast prestressed concrete floors and roofs in which only a thin concrete or cement and sand screed finish is needed to complete the structural unit, reference should be made to the recent tendency to use prestressed precast concrete shallow units which act as combined formwork and temporary support for a greater thickness of less expensive poured-in-place concrete which when set and effectively keyed to the prestressed soffit will carry the live load. In Fig. 9

Fig. 9.—Composite construction—precast prestressed soffit before placing of situ concrete

are shown units of this type developed by Mr. F. J. Samuely, and which have been fully described by him elsewhere.[11]

Miscellaneous Structures

High tensile alloy steel bars are also being used for an increasing variety of miscellaneous structures apart from buildings; in the case of the newly constructed wind tunnels for the N.A.E., designed by the Structural Engineering Division of the Ministry of Works, columns of cross-section shown in Fig. 10 were prestressed each with four bars of $\frac{7}{8}$ in. diameter.

The 40 ft. high hose drying column shown by Fig. 11 is an example of a member with several bars where stressing was done partially on two bars first and raised after similarly stressing the other pair of bars. In Fig. 12 which shows the stressing, it will be noticed spare adapters are used and left on the first pair of bars stressed to enable the jacks to be easily applied successively to pairs of bars and the stress raised a part at a time.

Circular Prestressed Tanks

Although for circular tanks the Preload method of wrapping with wires by a travelling apparatus is ap-

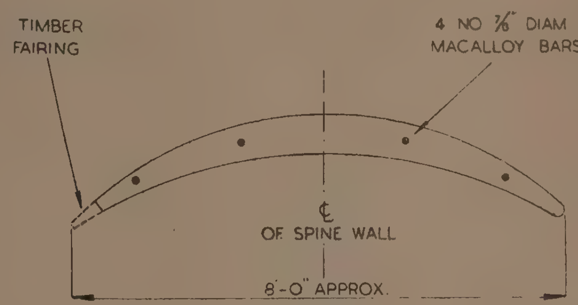


Fig. 10.—Details of columns in wind tunnel at Bedford

parently impossible to equal for large size tanks, for tanks of smaller diameter bars can be used very competitively because there is no plant required other than the standard type of prestressing jack. Fig. 13 shows a design for tanks in South Africa which have not yet been constructed. Naturally the saving in man hours in placing and stressing of the steel by the use of bars instead of wires can show in favour of the use of bars

Fig. 11.—40 ft. high hose-drying column being stressed. The roof beams shown were also prestressed on this system

where the comparison is with wires placed by hand. Where the wire is placed into position mechanically the rate of fixing the wire can come as low as 12 man hours per ton of wire fixed and stressed. For the circumferential stressing therefore, bars can only be expected to show a cost advantage over wire in the smaller sizes of tanks, although per ton of prestress the cost of bars is, as in other uses, generally lower. For the vertical prestressing of the tank walls, however, bars can be economically used for any size of tank, and Fig. 14 shows bars in tank walls, a number of large circular tanks

having been prestressed this way in the United States. These bars are coated and placed in position before the concrete and stressed from the top after hardening of the concrete. The technical disadvantages of "slip-rods" do not apply in a case like this because it is a practical impossibility to obtain more than the design hydrostatic pressure and increase of the moment beyond the design figure being virtually impossible, cracking of the concrete is impossible, as of course it should be with any water-retaining structure.

While in America the tendency is to coat bars where this can be done without loss of performance, the author favours wrapping in similar cases, the wrapping being preferably of a type which will compress slightly as the concrete shrinks without forming any grip on the bar. There are no practical difficulties in using the alloy steel bars as slip-rods, but grouting in of the bars is the recommended practice, and there is little difference in the cost.

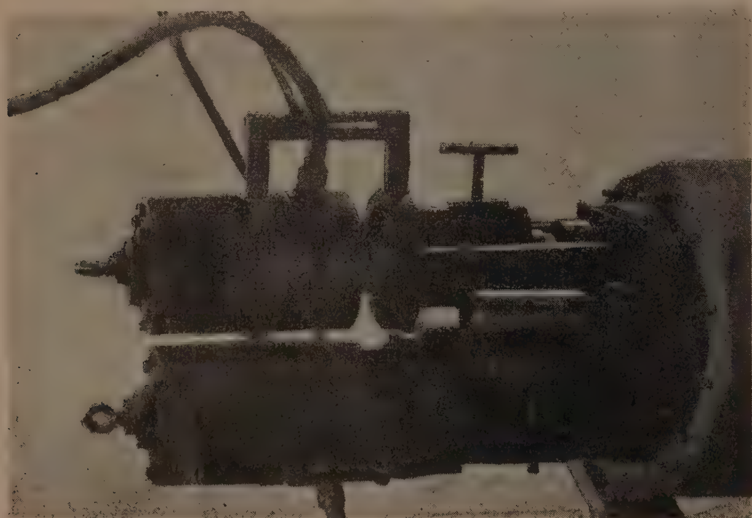


Fig. 12.—Stressing of the hose-drying column shown in Fig. 11

High Tensile Steel for Tie-bars

For tie bars an alloy steel having an ultimate tensile strength of 70 tons per sq. in. naturally has great possibilities, especially as there is no appreciable yield until about 0.85 of the ultimate strength. Assuming a working stress of the order of 32 tons per sq. in., this gives a working steel stress of practically four times the working stress on mild steel tie-bars, and in addition the use of the high efficiency couplers and nuts leads to further economy by reason of the avoidance of jumped-up ends or the alternative of designing the tie-bar on the net section after threading, which in the case of mild steel increases the weight of steel required by 25 per cent. and in that way results in the high tensile alloy steel saving about 80 per cent. of the weight.

In some applications this important economy, which allowing for the higher cost of the alloy steel is still considerable, is fully obtained. An example is the anchor ties to sheet piled walls, and in a case of this type the adjustment of length instead of being done by the usual turnbuckles is made by packing at one of the end anchorages when stressing.

In some cases, however, it is not desirable to use such a high stress because the extension under load is proportionately increased and in the case of ties to 2-hinge arches, or ties to structural steel portal frames of spans of the order of 130 ft., the use of high tensile ties without initial stress requires careful consideration.

Prestressed tie-bars can however be used very effectively, using a method thought by the author to have been first proposed by Mr. G. O. Kee, A.M.I.C.E. Where the ties are to be located in the soil the tie is located in a duct formed in the centre of a high strength concrete cylindrical casing and the tie stressed to an initial 30-40 tons per sq. in. giving a uniform prestress to the concrete, according to the circumstances, of the order of 1,000 to 2,000 lb. per sq. in. Subsequent variations in load in the tie due to loading on the arch or portal frame then only cause strains commensurate to the concrete stress, of any desired low value.

Prestressed Piles

Prestressed concrete piles are being increasingly used; they have been made for some time pre-tensioned on the long-line method, particularly in Sweden and this country, and post-tensioned on the Freyssinet system, particularly in France and in this country. In the latter case, the removal of the cone and anchorage at the pile head, for example if the pile is to be shortened or is damaged during driving, makes little or no difference to the performance of the pile, as the wires are grouted in and have a very considerable bond length. With pre-tensioned piles practically the same conditions apply and the bond length is an even greater number of steel diameters owing to the use of 14g and 12g wires.

One point of importance, however, is the resistance of the head to impact and the avoidance of destruction of the bond particularly at the head. The tendency may be to develop new types of pile heads where hard driving is expected.

Using high tensile alloy steel bars, different advantages and disadvantages arise.

In common with other types of prestressed piles, where the size of the pile is determined from stresses during handling and pitching, the superior strength of the prestressed piles enables smaller sections to be used, and in the case of very long piles besides an economy in first cost the driving equipment can be lighter and the driving itself slightly quicker. One obvious advantage is that piles can be made up in plain concrete sections and the whole stressed together with a single central bar, or a number of bars according to the type of pile and the design load, and couplers may be introduced for long piles or where driving is to be done with a limited height for pitching the pile. The use of couplers, however, is not normally necessary for piles less than 62 ft. long.

For marine use, piles prestressed with bars have in certain cases definite advantages due to the greater corrosion resistance of bars of large diameter. However, all prestressed piles have an advantage in this use due to the greater durability of "crackless" concrete. For a maximum ultimate strength in bending prestressed piles are preferably prestressed with the maximum possible lever arm, viz., the bars or wires are disposed around the periphery of the concrete section. However, for average conditions such a favourable combination of advantages over ordinary concrete piles, and low cost, is obtained by using a single bar at the centre and an axial prestress that this type of pile, either made in one piece or made up from sections, is likely to be most used in the near future.

Fig. 15 shows the detail of a prestressed pile of this type in which the resistance to bending stresses in pitching near to the picking up point is increased locally by a light cage of mild steel reinforcement. Piles of this type show a great economy in steel required and are superior to ordinary concrete piles to the extent that 10 in. by 10 in. prestressed piles can be compared with 13 in. by 13 in. ordinary concrete piles normally reinforced. Generally the author favours an initial prestress of the order of 1,000 lb. per sq. in. or slightly more, giving a final prestress of 0.84 of the initial prestress or about 840 lb. per sq. in. and in determining this factor no allowance is necessary with the alloy steel bars for end slip or creep of the steel. It is also of course possible to after completion of driving. However, in the author's opinion, taking account of all future possibilities during the life of the supported structure it does not seem generally desirable to remove the high tensile steel and leave the piles with no strength in bending.

Where piles have to be extended, couplers can be used to connect on mild steel or high tensile steel bars, and where it is desired to anchor down a structure this method obviously is useful.

Tubular Piles

Where large loads are to be carried or where piles of much longer than normal length are needed, tubular piles can be made up from a number of sections with the bars located in the thickness of the shell, as shown in Fig. 16. This method obviously is useful and likely to prove economical as well as giving a high degree of durability for marine use, for example, for piers of bridges, for heavy loads of quays and wharves, and for drilling platforms located off the sea coast. Joining on additional lengths of tubular piles by the use of couplers, may be done to save utilising extremely large pile frames.

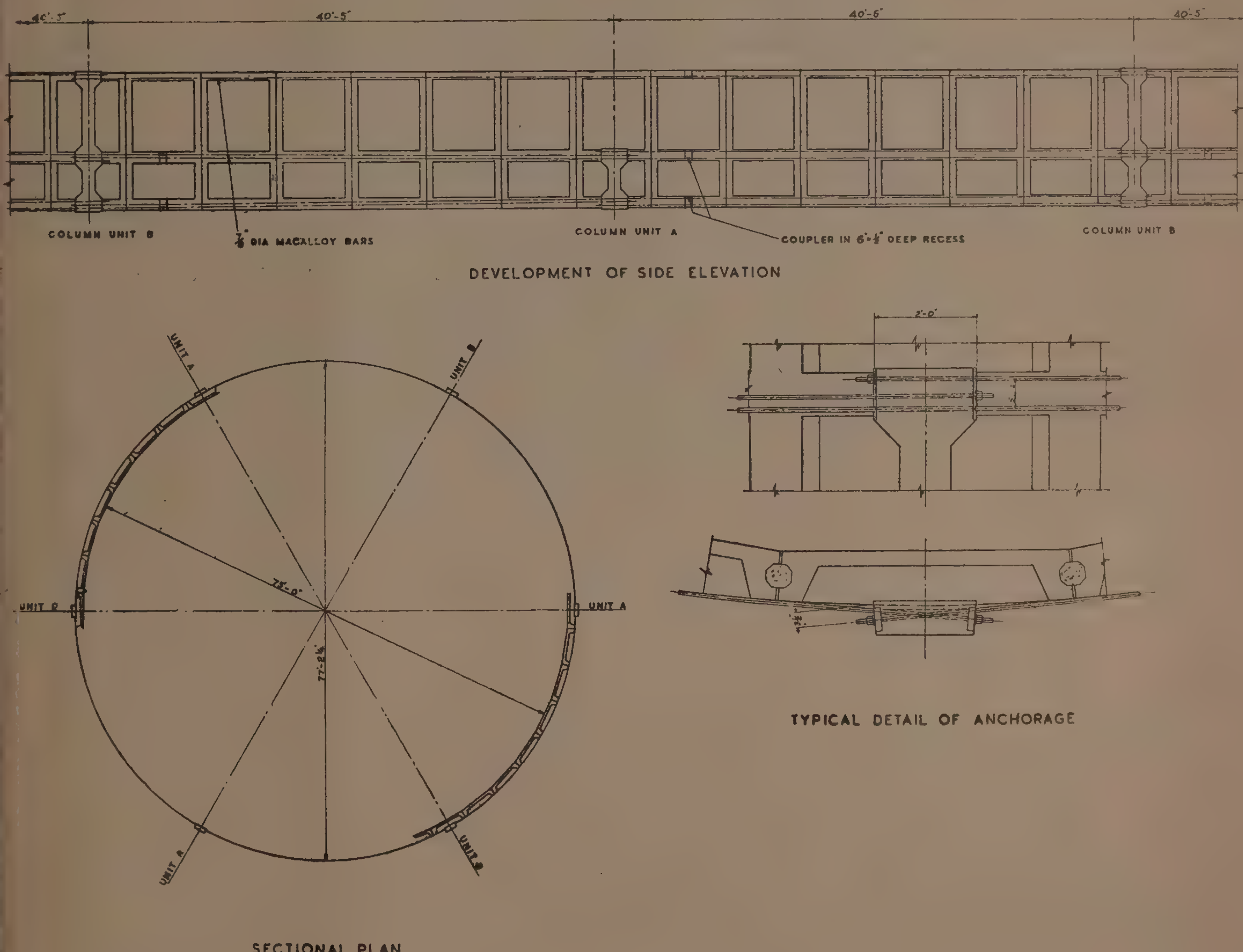


Fig. 13.—Method of using bars for circumferential prestressing of circular tanks

re-stress very easily to raise the prestress to the initial value.

Normally, extending the pile is easier than shortening it, and the making of piles longer than the correct length is generally avoided. However, where piles have to be shortened and particularly in the case of piles over 40 ft. long, the grout may be expected to hold the bar without losing the prestress on the concrete, seeing that there is a bond length generally of 400 diameters or more. In some cases of course, once the pile has been driven no bending stresses on the pile are expected subsequently, and following the author's recent paper in America[12] piles have been tried in California recovering the bars

Strengthening Existing Bridges

While the high tensile alloy steel was developed particularly for prestressing concrete, the use of it for strengthening existing bridges both of cast iron and steel is in

some cases economically very advantageous, and has the further advantage of generally permitting the strengthening to be done without putting the bridge temporarily out of use. Owing to the substantial diameters, the bars do not normally need more than ordinary paint protection. In Fig. 17 is shown the strengthening of a small road bridge in Yorkshire by the West Riding County Council, the existing beams being of cast iron and considered to be over-stressed. The high tensile alloy steel bars were added to these beams one each side of the web of the cast iron beams, and although only short lengths were involved, couplers were used to permit of easier placing in position. Other bridges have since been strengthened in this way, including some by British Railways.

Fig. 14.—Bars used as vertical prestressing units in Preload tanks in position against outer wall forms

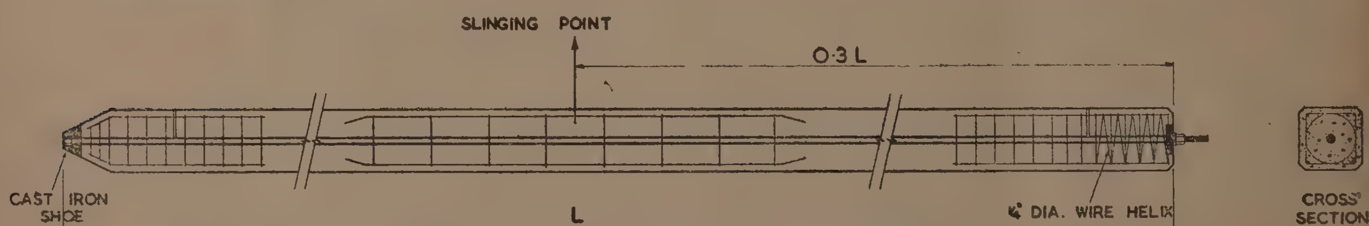


Fig. 15.—Detail of pile prestressed with single high tensile bar

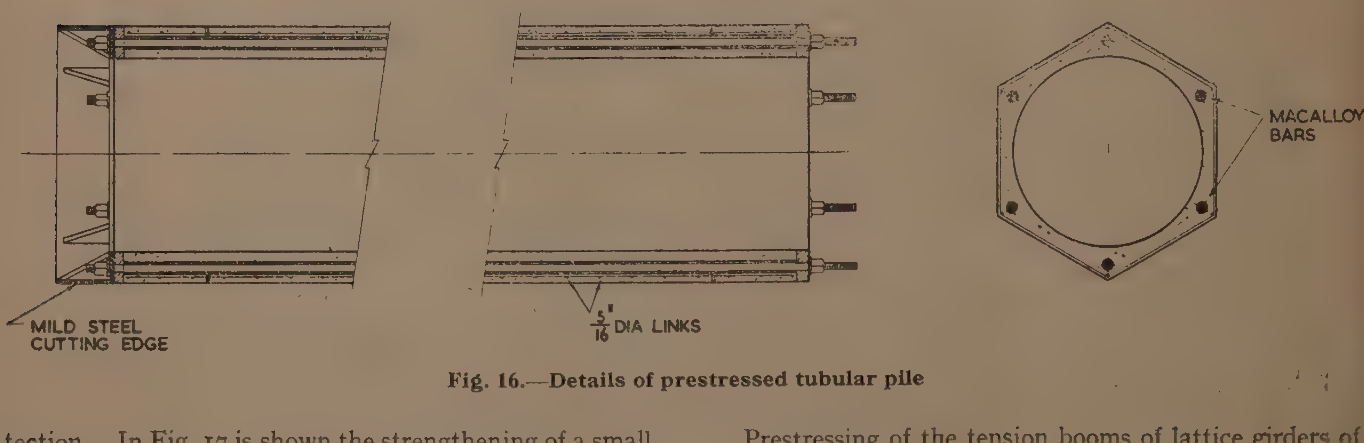


Fig. 16.—Details of prestressed tubular pile

Prestressing of the tension booms of lattice girders of structural steel bridges can be done where needed to increase their strength, particularly in cases where the compression boom may be strengthened correspondingly in some other way or does not need strengthening. Cases

of this type differ from the proposals recently made by Professor Magnel for achieving economy in steel by the prestressing of new lattice girders, in that there is no actual increase in deflection as compared with the existing bridge before strengthening, though in common with mild steel. The economy, the effectiveness and the possibility of doing the strengthening without serious interruption of the use of a bridge, however, all favour strengthening by prestressing and in the case of a lattice girder of 160 ft. span, the method of inserting the pre-

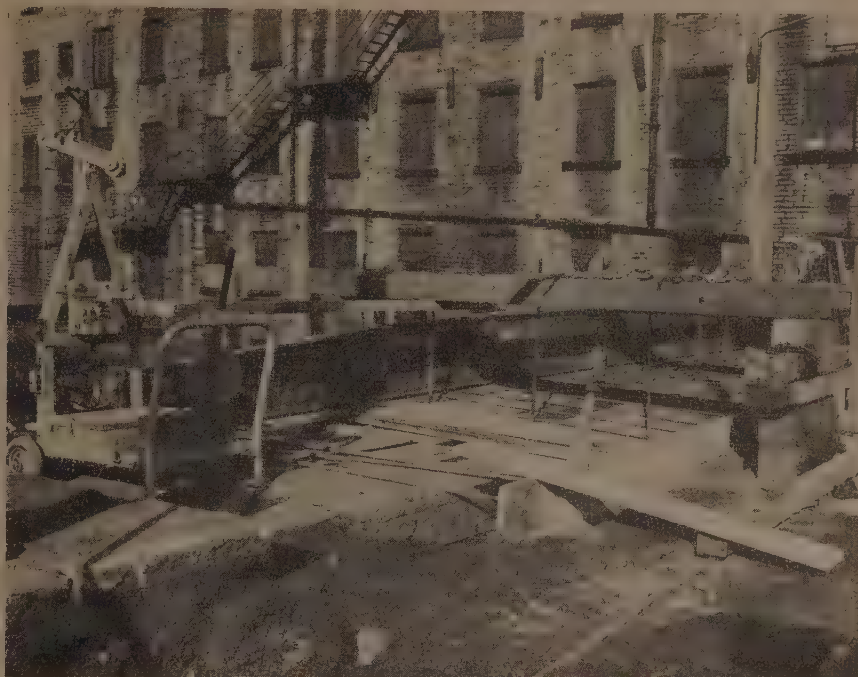


Fig. 17.—Cast iron beam bridge, strengthened by prestressing with high tensile bars

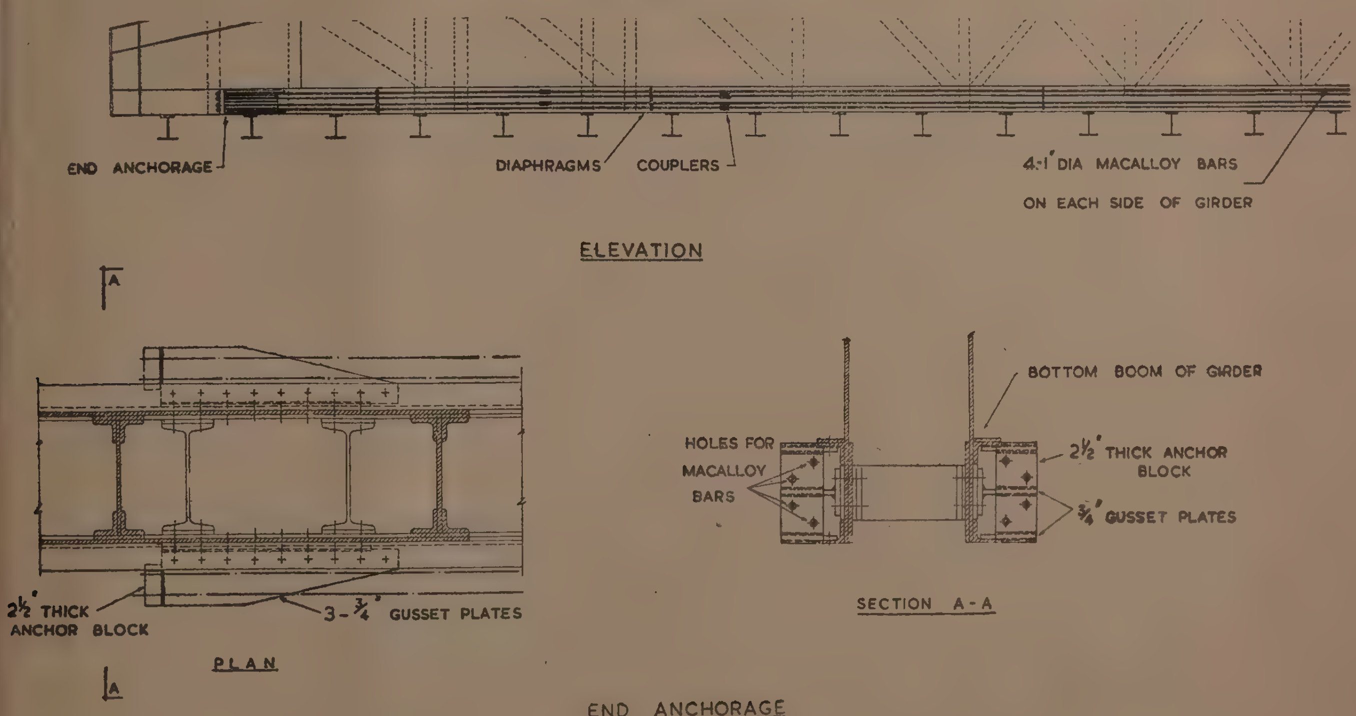


Fig. 18.—Method of strengthening tension boom of 160 ft. span lattice girder by British Railways, Western Region

the principles so well analysed by Professor Magnel in that paper,[13] the introduction of any prestressing steel whether it be bars or wire as part of the load-carrying structure does lead to an increase in the strain, and therefore the deflection, as compared with an equal amount of strengthening provided at a lower working stress, say by stressing steel is shown diagrammatically, by courtesy of British Railways, Western Region, in Fig. 18.

New Bridges

Developments in prestressed concrete bridges, even if restricted to the use of high tensile alloy steel bars, would

alone justify a paper of unusual length, and it will be desirable therefore merely to touch upon a few matters of technical importance and include references to a few examples of bridges recently completed or now in hand in which these points arise. Generally speaking, prestressed concrete is inherently one of the best materials obtainable for bridge construction, and this of course arises primarily from the advantages of "crackless" concrete and the very low maintenance and long life that we have every reason to believe will follow from its use.

Apart from that, however, in another major respect prestressed concrete is particularly suitable for bridge construction. This is owing to its great resistance to fatigue arising from repetitions of application of live load. This may seem curious to metallurgists and those civil engineers familiar with the reduced resistance to fatigue of high tensile steels such as 0.8 Carbon steel cold-drawn wire. The alloy steel bars having a lower carbon content and a greater ultimate elongation have some advantages over plain carbon cold-drawn wire in this respect, but the reason why prestressed concrete has a greater resistance to fatigue than either structural steel (riveted or welded) bridges, or ordinary reinforced concrete, is that the variations in stress in the steel due to application of the live load are, by comparison and in fact, extremely small, while in other forms of construction the live load often doubles the steel stress. In a typical case, the change of steel stress in a bar initially tensioned to 42 tons per sq. in. is only of the order of 1 to 1½ tons per sq. in., and in considering this it must not be overlooked that the loss of prestress by shrinkage and creep of the concrete, averaging 16 per cent., will normally have lowered the tension of the steel before the bridge is brought into use by some 3 or 4 tons per sq. in., so that applications of the full design live load do not raise the steel stress even back to what it was when initially tensioned. Further, this variation of stress occurs at a point of maximum moment, which may for example be the mid-span, and at the end anchorages with the bars grouted in there cannot be any alteration of stress at all, unless the grout should be so imperfectly done that it will not even transfer 1 ton per sq. in. steel stress in a length from mid-span to the end, which may vary from 100 to several hundred bar diameters. If the grout should fail, or if it should have been omitted, there will be variation of stress in the end anchorages at each application of the live load, but for uniformly distributed load this will only be two-thirds the stress variation previously mentioned, i.e., of the order of two-thirds of 1 ton per sq. in.

Fig. 19.—50 ft. span footbridge constructed by Lindsay County Council

Fig. 20.—Precast sections for 34 ft. span footbridge for Thames Conservancy

The range of the alteration of steel stress is of course m times the variation in the stress of the concrete immediately adjacent, m being taken at the value for instantaneous loading, and in a typical case, assuming the dead load is acting continuously, the variation in concrete stress at the level of the bars at the point of maximum moment does not generally exceed 500 lb. per sq. in. Young's modulus of the alloy steel is close to 25 million and does not vary from this below a steel stress of 60 tons per sq. in. The instantaneous value of E for concrete subjected to repeated alterations of live load, such as train axles passing over a short span bridge, may be close to 6×10^6 lb. per sq. in. giving a value of m

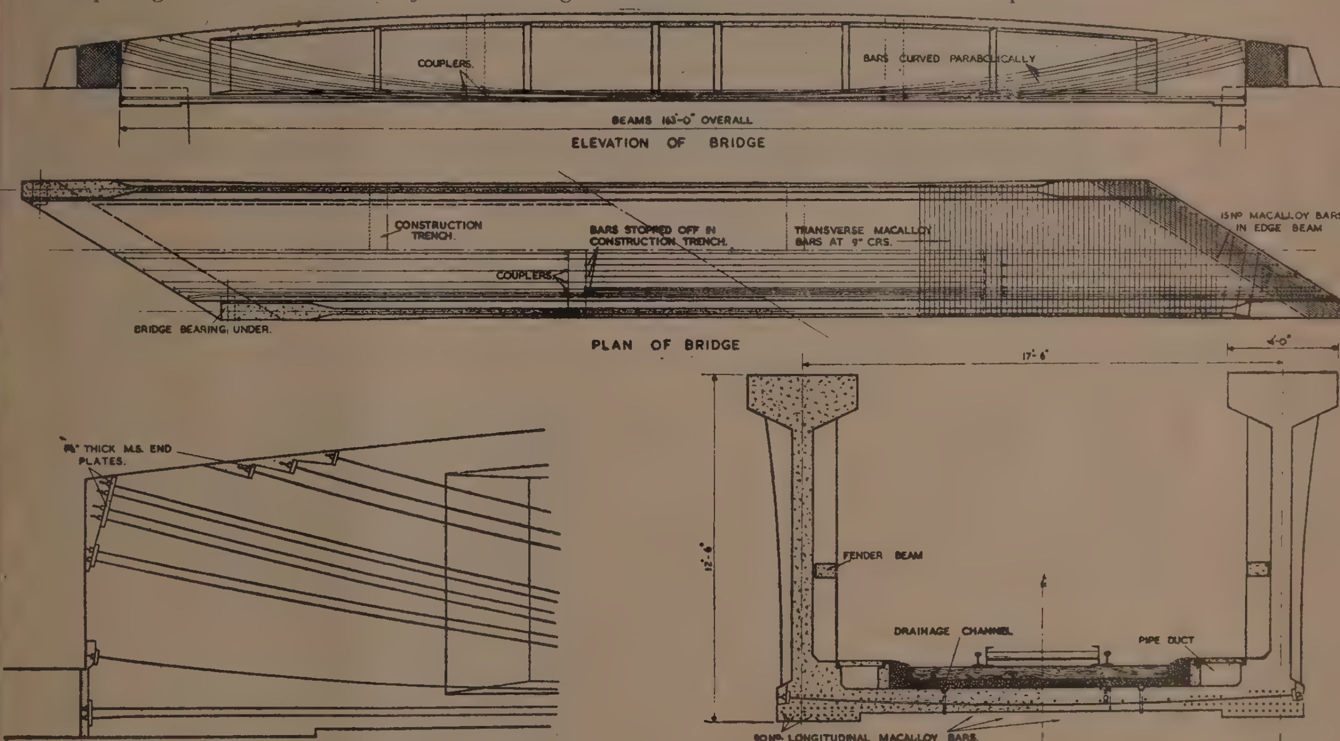
Fig. 21.—Typical cross-section of steel girder and precast prestressed concrete deck bridge

of slightly over 4, and taking 5 as a safe figure the variation in steel stress at mid-span will usually be of the order of 2,500 lb. per sq. in.

There is every reason to believe therefore that prestressed concrete when properly designed will give a very high degree of resistance to fatigue, and as far as the author is aware there is no evidence that has come to hand since the introduction of prestressed concrete to bridges on which any information has been obtained to throw any doubt on this.

Nevertheless it is, in the author's opinion, desirable to grout in all bars in bridges to ensure that variations of steel stress are not needlessly transferred to end anchorages even although by the use of slip-rods or uncased bars the variation in stress in this case is generally of the order of two-thirds (or occasionally slightly less) of the change of steel stress at mid-span of fully bonded bars. Moreover, in many cases economy can be obtained and the cost of forming ducts avoided by using exposed bars at the sides of beams through all the middle part of large spans and disposing the bars so that they enter into grouted ducts in the 10 ft. or thereabouts at each end of long spans. This is of course different to the method which has been used in Germany by Finsterwalder in trussing concrete members and applying a prestress by means of bars generally well below the level of the concrete.

Fig. 22. - Precast prestressed slab type of bridge being used by British Railways, Western Region, for short spans



Details of Beam End — All Macalloy Bars in Bridge are $1\frac{1}{8}$ in. dia. in $1\frac{3}{8}$ in. dia. cavities — Bridge Section

Fig. 23.—160 ft. span through-type underline railway bridge over the River Don, near Rotherham

Fig. 24.—The River Don railway bridge under construction showing soft rubber and pneumatic tubes

Prestressed concrete enables very shallow constructional depths, particularly in the case of small bridges of prestressed slab type, subject however to the desirability to avoid high span/depth ratios where the natural frequency is close to that of likely applications of the live load.

Fig. 19 shows a typical example of a footbridge and the slender construction possible. The depth of construction is only 9 in. for a span of 50 ft. For a towpath bridge on the River Thames Fig. 20 shows a precast prestressed bridge of the double cantilever type prestressed preparatory to being taken to the site. Precast prestressed concrete slabs have been recently brought into use by British Railways, Western Region, for the renewal of existing bridges in cases where the work has to be

carried out in the minimum time. Fig. 21 gives, by courtesy of British Railways, a typical cross-section of a under-line bridge in which the steel girders are provided with connections for bolting in position individual

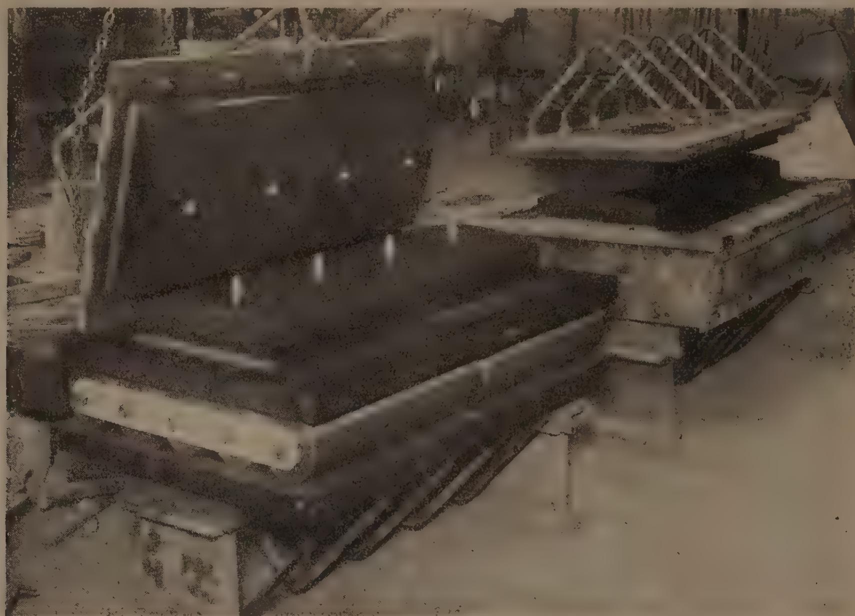


Fig. 25.—Rolling and rocker bearings for one end of the River Don bridge

slabs as they are placed in position by a locomotive crane. In some other cases British Railways, Western Region, are renewing short span bridges by precast prestressed slabs spanning from abutment to abutment, as shown in

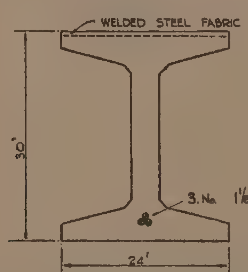


Fig. 26.—Cross section at mid-span of one of the 500 beams used for Leyton Marshes Culvert

Fig. 27.—Leyton Marshes Culvert—Beams in position

Fig. 22, and with this type the kerb is precast with the deck. Where prestressed slabs have short spans of this order, it is of course most desirable to ensure complete bonding of the bars by grouting after stressing at least one week before any applications of live load, and in this particular case, grouting is done at the precasting works well prior to handling for despatch to site. Where slabs of this type have a number of prestressing bars the order

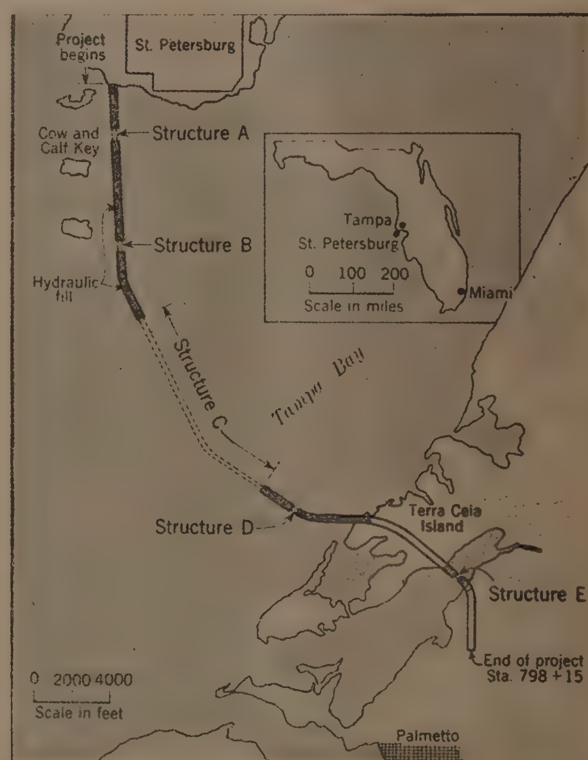


Fig. 28.—Lower Tampa Bay Bridge, Florida—Location plan

of stressing needs to be arranged to avoid any large temporary variations in stress across the section, and in addition it is generally necessary to provide transverse reinforcement, which may often be cold drawn welded mesh, both top and bottom of the slab to take the tensile stress across the slab produced by the longitudinal pre-stress. This transverse reinforcement is normally determined from the prestress and Poisson's ratio, but in

cases where there is also restraint to shrinkage this can be provided for at the same time as it is of contrary sense.

160 ft. span Prestressed Rail Bridge, Rotherham

In the example shown by Fig. 23 of the 160 ft. span single track railway bridge over the River Don near Rotherham, although additional side clearance is provided, the deck slab is only $10\frac{1}{2}$ in. thick, and this may be compared with similar deck slabs of ordinary reinforced concrete with which the author has been concerned earlier, of the order of 18 in., both being on the basis of 20 unit loading and usual impact allowances.

This is possibly the largest span prestressed rail bridge constructed to date. In this particular case the necessity on the one hand of allowing for high flood water level, and on the other the avoidance of raising the railway level, dictated the through type of prestressed girders used. Normally, of course, it would have been more economical to have been able to place the main beams below the deck slab. Nevertheless, the estimated saving in cost of the whole bridge, including the pile foundations and abutments, is not less than £7,000 by using prestressed concrete instead of structural steel, and although the piling and the abutments require the same type of construction in either case, the time for completion at the time this contract was put in hand was in favour of prestressed concrete ; in normal times it would probably have been in favour of structural steel lattice or plate girders. A view of the deck slab ready for concreting, and the pneumatic rubber tubes forming the ducts for the main bars in the girders and the soft rubber tubes for the transverse bars are seen in Fig. 24. The combined rocker and roller bearings shown by Fig. 25 are to carry a vertical load of 300 tons and allow for the $\frac{3}{4}$ in. movement due to stressing, apart from the usual temperature movement.

Leyton Marshes 1,000 ft. Culvert

For roofing over a length of slightly more than 1,000 ft. of the flood diversion channel of the River Lea in Essex, some 500 beams 57 ft. long placed side by side are being used, designed to take a thin earth cover and full Ministry of Transport highway loading. Transverse prestress is used connecting every 20 beams together with an expansion joint between each block.

This construction was an alternative to normal reinforced concrete with a centre support halving the span of the beams, and as compared with the reinforced concrete on half span the tenders were practically the same. By courtesy of the Lee Conservancy Catchment Board, a cross-section of one of the beams is shown in Fig. 26 and of the beams placed in position in Fig. 27. The initial stress was the usual value of 42 tons per sq. in. and the concrete specified in this case, slightly lower than the usual 28 day cube strength requirement, at 5,500 lb. per sq. in., the design resisting moment being 5,200,000 lb. in. One of these beams was tested by the Cement and Concrete Association, and the test is being described elsewhere, so it is probably only of immediate interest now to mention that the ultimate strength was almost exactly as calculated after allowing for the much superior 28 day concrete cube strength of around 7,100 lb. per sq. in. which was actually obtained. The test verified the ultimate strength as equivalent to dead load plus 2.4 times the full M.o.T. highway loading and the failure took place in the concrete as expected, none of the bars being broken, the end anchorages being, as expected, quite unaffected. One interesting point revealed by the test was the completeness of the grouting in of the bars after the stressing, and the adherence of the grout to the main concrete.

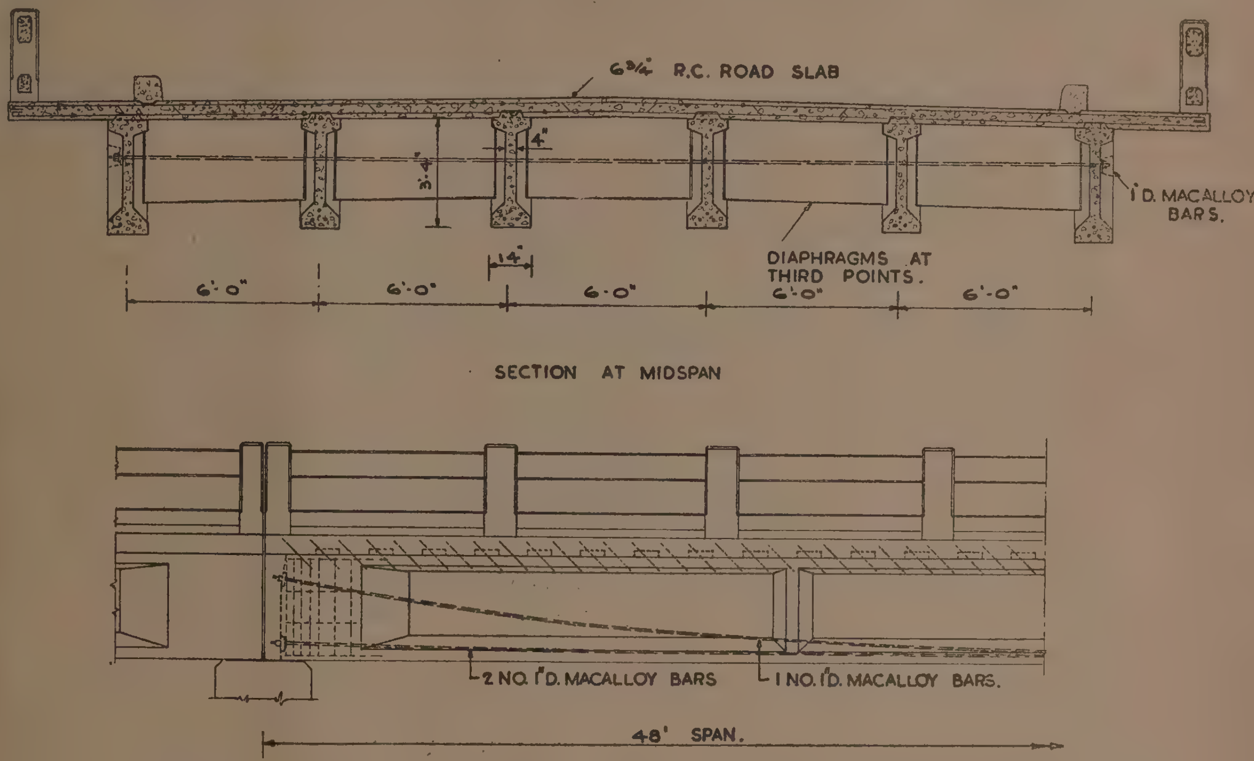


Fig. 29.—Lower Tampa Bay Bridge, Florida—details of prestressed construction

Tampa Bay Bridge

Of various projects abroad, need to limit the length of this paper has necessitated only trifling reference. The 15 mile long toll bridge across Lower Tampa Bay in Florida is, however, of more than usual interest. The first section of about four miles was designed and contracts arranged before prestressed concrete was

considered. The bridge which (as shown by Fig. 28), crosses the sea for most of its length, was originally designed to have 36 ft. ordinary reinforced concrete spans supported by pile trestles for the majority of its length, with steel lattice girders (for a length of 5,621 ft. including one of 864 ft. span) at the main ship crossing, at which point the headroom is 150 ft. At another point a double leaf bascule bridge is provided. After the contract was placed for the main Northern section, alternative designs were invited in prestressed concrete for the Southern main section of the bridge covering one length of 10,896 ft. and another of 585 ft. totalling slightly more than three miles, which with another

[2]Lee-McCall system of Prestressed Concrete. ENGINEERING. April, 1950.

[3]Prestressed Concrete for Docks and Marine Works. THE DOCK AND HARBOUR AUTHORITY. May, 1950.

[4]A New System of Prestressed Concrete. CIVIL ENGINEERING. May, 1950.

[5]Prestressed Concrete Developments at the Field Test Unit Thatched Barn, by O. J. Masterman, B.Eng., A.M.I.C.E., A.M.I.Struct.E. THE STRUCTURAL ENGINEER. October, 1950.

[6]Tests on prestressed concrete beams with alloy steel bars, by A. D. Ross, B.Sc., Ph.D., M.I.C.E., F.R.S.E. MAGAZINE OF CONCRETE RESEARCH. No. 7. August, 1951.

[7]Prestressed Concrete Structure at an Exhibition. CONCRETE AND CONSTRUCTIONAL ENGINEERING. November, 1950.

[8]CANADIAN BUILDER. July, 1951.

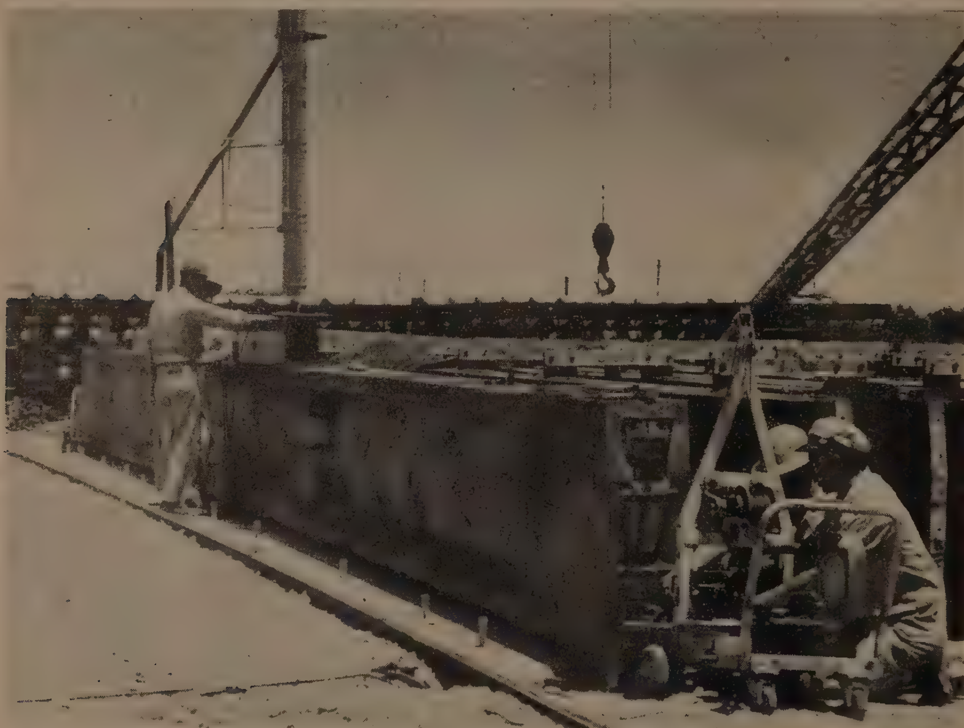


Fig. 30.—Lower Tampa Bay Bridge, Florida—Test girder during stressing operation

shorter section of 672 ft. on the South side of Terra Cela Island, made the total length for which prestressed concrete was considered 17,400 ft. Of the three alternative prestressed designs, that shown by Fig. 29 was adopted, based on the original reinforced concrete design of 36 ft. used in Sections A and B at an estimated nett saving over the normal reinforced concrete scheme of $144,000. This design was based on trestle centres and spans of 48 ft. so that with six prestressed beams to each span the total number of beams is 2,178. It will be seen from Fig. 29 that a transverse prestress is provided. The prestressed beams were designed to support the deck slab until it had hardened and the combined construction designed to take U.S. H-20 loading with a calculated factor of safety in terms of live load of 3.9, the dead loading being considered unalterable.

One of the first beams made was tested to destruction and showed a factor of safety, in terms of the live load, of 5 and another of the beams was unaffected by a shear test which was equal to four times the design live load, plus unit dead load.

When completed this is expected to be the longest prestressed concrete structure in the world.

References

[1]Steel Alloy Bars used in Prestressed Concrete. CONCRETE AND CONSTRUCTIONAL ENGINEERING. April, 1950.

[9]ROAD CONSTRUCTION. 1952.

[10]HORMIGON ELASTICO. December, 1951.

[11]Some recent experience in Composite Precast and *in situ* Concrete Construction, with particular reference to Prestressing, by F. J. Samuely, B.Sc.(Eng.), A.M.I.C.E., Institution of Civil Engineers, February, 1952.

[12]Prestressed concrete using High Tensile Alloy Steel Bars, by D. Lee, Paper No. 2,3, First U.S. Conference on Prestressed Concrete, Massachusetts Institute of Technology, Cambridge, Mass., August, 1951.

[13]Prestressed Steel Structures, by G. Magnel. STRUCTURAL ENGINEER. November, 1950.

[14]Discussion by R. E. Sadler on Prestressed Concrete Applied to the Construction of Railway Bridges and other works, by Arthur Dean, M.Sc.(Eng.), M.I.C.E., Institution of Civil Engineers. May 8th, 1951.

Acknowledgements

(additional to those given in the text)

Fig. 7. British Railways, Eastern Region, and Institution of Civil Engineers.

Fig. 10. Mr. G. A. Gardner, Superintending Structural Engineer, Ministry of Works, and Messrs. J. L. Kier & Co.

Figs. 11 and 12. Mr. G. A. Gardner, Ministry of Works, and Bristol Stone and Concrete Co.

Fig. 14. The Preload Company, New York.

Fig. 20. R. V. Stock Esq., M.C., M.I.C.E. Chief Engineer, Thames Conservancy, and Cowley Concrete Co.

Fig. 27. Marshall Nixon Esq., M.B.E., A.M.I.C.E., Chief Engineer, Lee Conservancy Catchment Board, and Concrete Piling Ltd.

Institution Notices and Proceedings

GENERAL MEETING

A General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London S.W.1. on Thursday, October 9th, 1952, at 6 p.m., when the Presidential Address for the Session 1952-1953 was given by Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E.

The Secretary read a letter from the retiring President, Mr. Walter C. Andrews, O.B.E., M.I.C.E., M.I.Struct.E. expressing his regret at being prevented by illness from attending the meeting and conveying his good wishes to Mr. Granter for a happy year of office.

Mr. J. E. Swindlehurst, O.B.E., M.A., M.I.C.E. (Immediate Past President) was in the Chair.

It was agreed that a message be conveyed to the President wishing him a speedy recovery.

The Chairman welcomed the guests who were present and presented the following medals and awards for the Session 1951-1952 :—

Institution Bronze Medal to Lt. Col. G. W. Kirkland, M.B.E. (Member of Council) for a paper on " Design and Construction of a Large Span Pre-stressed Concrete Shell Roof."

Research Medal to Dr. P. W. Abeles for a paper on " Breaking Tests on Three Full Size Pre-stressed Concrete Beams."

Research Diploma to Mr. W. Shearer Smith for a paper on " Cold Formed Sections in Structural Practice with a proposed Design Specification."

Research Prize to Mr. O. J. Masterman, for a paper on " Pre-stressed Concrete Developments at the Field Test Unit, Thatched Barn."

After some introductory remarks regarding Mr. Granter's career and his work for the Institution, the Chairman invested him with the Presidential Badge.

Mr. Granter then took the Chair and called on Mr. G. B. R. Pimm, M.I.C.E., M.I.Struct.E., (Past President) and Lt. Col. H. S. Rogers, C.M.G., D.S.O., M.I.Struct.E., (Past President) to propose and second a vote of thanks to Mr. Walter C. Andrews for his work as President of the Institution during the Session 1951-1952.

The vote of thank was carried and Mr. Granter then gave the Presidential Address for the Session, which is printed in this issue.

At the conclusion of the Address, a vote of thanks to the President was proposed by Mr. F. S. Snow, M.I.C.E., M.I.Struct.E., (Past President) and seconded by Mr. W. K. Wallace, C.B.E., M.I.C.E., M.I.Struct.E., (Past President). This was carried by acclamation. The President responded and the proceedings then terminated.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution was held at 11, Upper Belgrave Street, London, S.W.1., on Thursday, October 23rd, 1952, at 5.55 p.m., Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E., (President), in the Chair.

The Minutes of the Ordinary General Meetings held on May 22nd and June 26th, 1952, as published in the Journal, were taken as read, were confirmed and signed.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Graduates

Ball, Thomas Christopher Gann, of Cuffley, Herts.
Czarnecki, Jan, of London.
Evans, Robert Edgar, of London.
Gach, Jerzy, of London.
Holocher, Roman Wieslaw, of London.
Janka, Kazimierz, of London.
Jenkins, Roy George Henry, of Bristol.
Kucharski, Albin Michal, of London.
Rzadkiewicz, Stanislaw, of London.
Slowikowski, Stanislaw, of London.
Thei, Arthur Abraham, of London.
Wadon, Jan Jerzy, of London.
Wotton, Frederick Ernest, of Margate, Kent.

Associate-Members

Anderson, Robert, of Thornliebank, Renfrewshire.
Athavale, Shrikrishna Gangadhar, B.Sc., B.E. Bombay, of London.
Barnes, Robert Gerald, B.Sc.(Civil), Manchester of London.
Baron, John William, of Stafford.
Bennett, Colin Joseph, of Watford, Herts.
Binns, Robert Derek, B.Sc.(Eng.) London, of Harrow, Middlesex.
Black, Robert, of Motherwell, Lanarks.
Chappell, Donald, of Horley, Surrey.
Clark, George Hyde, of London.
Creet, John George, B.Sc.(Eng.) London, of London.
Day, James Archie, of Banstead, Surrey.
Dennard, Bertram, of Risley, Lancs.
De Souza, Ronald Cajetan, B.E.(Civil) Bombay, of Twickenham, Middlesex.
Elsey, Morris Burfield, of Sidcup, Kent.
Evans, David Edmund, of London.
Gawlinski, Andrzej Wilhelm, of Manchester.
Harvey, John Alan, B.Sc. (Eng.) London, A.M.I.C.E., A.C.G.I. of Oxford.
Landau, Robert Elkan, B.Sc. (Eng.) London, of London.
Langford, Lewis, B.Eng. Sheffield, of Stafford.
Martin, William George, of Bromborough, Cheshire.
Ovens, Hubert, A.M.I.Mun.E., of Barking, Essex.
Shields, Douglas John, B.E. (Civil) Bristol, of Loughborough, Leics.
Snowden, George Williams, of Middlesex.

Transfers

Students to Graduates

Adams, Peter Henry, of Westcliffe-on-Sea, Essex.
Bailey, John, of Warrington, Lancs.
Barlow, Eric, of Stockport, Cheshire.
Barnes, Victor, of London.
Bateman, Douglas James, of Chessington, Surrey.
Chilvers, Alan, of Newcastle upon Tyne.
Coleman, Peter Stanley, of Hutton, Essex.
Conway, Gerald Ernest, of Welling, Kent.
De Penning, Jean Charles, of London.
Ellis, Kenneth George, of London.
Farley, Michael George Gilbert, of Plymouth, Devon.
Finch, Cecil William, of Greenford, Middlesex.
Holmes, Gordon Victor, of Birmingham.
Iwanski, Zygmunt, of London.
Jones, Royston William, of Caerphilly, Glam.
Kirkman, Cyril John, of New Barnet, Herts.
Krol, Tadeusz, of London.
Lenartowicz, Witold, of London.
Llewellyn, Peter Charles, of Shoeburyness, Essex.
Mason, John Francis, of Borrowash, Derbyshire.
Masters, Patrick Anthony, of Derby.
Noble, Colin, of Dewsbury, Yorkshire.
Nuttall, Peter, of Southall, Middlesex.
Orpiszak, Boleslaw, of London.

ROBERTS, Geoffrey Russell, of Chepstow, Mon.
ROSE, Capt. Charles Frederick, R.E., of Hove, Sussex.
STACEY, Donald William, of Dagenham, Essex.
STUDZINSKI, Stanislaw Henryk, of London.
WARRINER, Harold Maurice, of Salford.

Graduates to Associate-Members

BAILEY, Stanley, of London.
BARCHAM, William John, of South Ockendon, Essex.
BAYLEY, Jack Borough, of Purley, Surrey.
BENNETT, Robert Walter, of Harrow Weald, Middlesex.
BILLINGTON, Roger Dean, of Nottingham.
BOND, Peter Henry, of London.
BOOTH, Robert Stuart, of Birmingham.
BROUGH, Derek Fletcher, of Manchester.
CAMPBELL, Gordon Arnison, M.Sc. (Civil) Leeds, of Leeds, Yorks.
CHAPMAN, Kenneth Geoffrey, B.Sc. (Civil) Leeds, of Harrogate, Yorks.
CROSTHWAITE, Donald Rothery, B.A. Cambridge, of London.
CUSSONS, Stanley Harold, of Stockton-on-Tees, Co. Durham.
DAVEY, Kenneth Clive, of Scunthorpe, Lincs.
DEMBINSKI, Maciej Jerzy Jozef, B.Sc. (Eng.) London, of Middlesbrough, Yorks.
EDWARDS, Jack Alfred, of London.
EDWARDS, Philip Birchall, of London.
ELLIS, Ryan, B.Sc. (Civil) Belfast, of Belfast.
FARNABY, John Eric, of London.
FULLER, Edward Thomas, of Orpington, Kent.
GORECKI, Alexander, of Manchester.
HARVEY, Wreyford Frank Petrie, of Hove, Sussex.
HILL, John Worsley, B.Sc. (Tech.) Manchester, of Manchester.
JONES, Ivor Philip Thompson, B.Sc. (Eng.) London, of Edgbaston, Birmingham.
KADIANI, Fida Husein Ghulamali, B.E. (Civil) Bombay, of Saddar, Karachi, Pakistan.
KAY, Marzell, M.Sc. Birmingham, of Leamington Spa, Warwickshire.
MALE, Edwin Talbot, of Lowestoft.
MORRAY-JONES, Robert Henry, B.Sc. (Civil) Birmingham of Coventry.
NABI, Gamil Abdel, M.Sc. London, of Manchester.
NAYLOR, Gerald Wilfrid, B.Sc. (Civil) Leeds, of Harpenden, Herts.
NORFOLK, John Duncan, A.M.I.Mech.E. of Portsmouth, Hants.
NORMAN, John Leonard, of London.
PAGE, Frank Arthur, B.Sc. (Eng.) London, of Colchester, Essex.
ROWLEY, Noel, of Derby.
STAIG, Alan Franklin, B.Sc. (Eng.) London, of Reading, Berks.
SUDGEN, Derek Taylor, of Harrow, Middlesex.
TAYLOR, Gerald, of Reading, Berks.
THOMPSON, Peter Charles, of Sonning Common, Oxford.
THORLEY, Walter, of Stockport, Lancs.
TINSLEY, Peter Hugh, B.A. (Hons.) Cambridge, of Risley, Lancs.
TOWLER, Robert Jack, of Banstead, Surrey.
WADDY, Harry Francis, B.Sc. (Eng.) London, of Bromley, Kent.
WEEKS, Peter Charles, B.Sc. (Eng.) London, of Plymouth, Devon.
WILLIAMS, Keith Henry Glyn, of Erith, Kent.
WOOD, Peter Buckley, B.Sc. (Tech.) Manchester, of Salford, Lancs.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, December 11th, 1952

Ordinary Meeting, 6 p.m. Mr. Donovan H. Lee, B.Sc., M.I.C.E., M.I.Mech.E., M.Am.Soc.C.E. (Member of Council) will give a paper on "Prestressed Concrete Bridges and other Structures."

Thursday, December 18th, 1952

Ordinary General Meeting for the election of members only, 5 p.m.

Thursday, January 8th, 1953

Joint Meeting with the Reinforced Concrete Association, at 6 p.m., when Colonel A. R. Mais, O.B.E., T.D., and Mr. A. C. Little will give a paper on "The Construction of Eight Prestressed Concrete Tanks."

Thursday, January 22nd, 1953

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. B. A. E. Hiley, M.I.C.E., (Member of Council) will give a paper on "Electricity Generating Stations."

Thursday, February 12th, 1953

Ordinary Meeting, 6 p.m., when Dr. F. G. Thomas, M.I.C.E. (Member of Council) will give a paper on "The Strength of Brickwork."

Thursday, February 26th, 1953

Ordinary General Meeting, for the election of members, 5.55 p.m. followed by an Ordinary Meeting at 6 p.m., when Mr. P. L. Capper, T.D., M.Sc., A.M.I.C.E., (Member of Council) will give a paper on "Soil Mechanics in Relation to Structural Engineering."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

OBITUARY

The Council regret to announce the death on October 16th, of Mr. Albert S. Spencer (Retired Member). Mr. Spencer was 71 and was elected to Membership of the Institution in 1922. He was a former Council member and was Chairman of the Lancashire and Cheshire Branch in 1922-1923.

HONOURS AND AWARDS

In offering their sincere congratulations to the following member on the distinction recently conferred upon him, the Council feel that they are also expressing the good wishes of the Institution.

ORDER OF THE BRITISH EMPIRE—M.B.E.
Mr. F. J. Green (Member).

MACLACHLAN LECTURE COMPETITION, 1953

The closing date for the receipt of entries for the next MacLachlan Lecture Competition will be Tuesday, March 31st, 1953. Particulars of the Competition are as follows :—

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering so long as in every second year the subject shall be confined to steel structures. (1953 is one of these years.)

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publica-

tion in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s. od.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer these sums to the Research Fund of the Institution.

DRURY MEDAL AWARD

The fourth competition for the above award will take place in 1953. The subject is the design of the structure of a new factory building. The material of construction is entirely at the choice of the competitor. The competition has been designed to encourage ingenuity of structural arrangement. Economy in the use of steel is an important feature of this year's competition.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary ; envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st, 1953.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less that five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

LONDON GRADUATES AND STUDENTS SECTION

The next meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1., on Tuesday, December 9th, at 6 p.m., when a film on " Takoradi Dry Dock " will be shown by courtesy of Messrs. Taylor Woodrow. This will be followed by a talk on the project by a representative of the contractors.

Hon. Secretary : C. Allen Brown, 43, Coolgardie Avenue, Highams Park, London, E.4.

CORRIGENDA

September issue.

Page 224, col. 1, lines 7 and 8. The equation should read :—

$$M_{ult} = .9 \times d \times A_s \times t_{ult}$$

line 12. The equation should read :—

$$d = \tfrac{2}{3}D$$

March issue.

Page 71. The initials of the reviewer of the book " Moving Forms " by L. E. Hunter should read " C. P."

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Tuesday, January 13th, 1953

Joint Meeting with the Institute of Welding, Liverpool and District Branch when the 1951 Larke Medal Paper on " Continuous Welded Structures, Abbey Works, Port Talbot " will be given by Mr. W. R. Atkins, B.Sc., M.I.C.E., M.Inst.W., at the Liverpool College of Technology at 7 p.m.

Wednesday, January 28th, 1953

Mr. Ronald Oates (Graduate) on " The Structural Design of the Medieval Cathedral."

Tuesday, February 24th, 1953

Mr. P. W. Rowe, B.Sc., Ph.D., A.M.I.C.E. on " Developments in the Design of Sheet Pile Walls."

All meetings, unless otherwise stated will be held in the Reynolds Hall, College of Technology, Manchester at 6.30 p.m., preceded by tea at 5.45 p.m.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 24, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Friday, January 23rd, 1953

Dr. K. Hajnal-Konyi, A.M.I.C.E. (Member) on " Recent Applications of Shell Concrete Construction in England and Wales."

Tuesday, February 17th, 1953

Mr. O. W. Jones, B.Sc., A.M.I.C.E. (Member) on " Reinforced Concrete Foundations and Structures for Blast Furnaces and Materials and Handling Plant at Shotton, Nr. Chester " at Kings Hall, Queen Street, Derby at 7 p.m.

Friday, February 25th, 1953

Mr. N. T. Grant on " Experiences with Concrete."

All meetings, except where otherwise stated, will be held in the James Watt Memorial Institute, Birmingham, at 6 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

MIDLAND COUNTIES BRANCH

The following meeting has been arranged :—

Friday, January 30th, 1953

" Further Recent Midland Structures." Descriptions of Rebuilding of Marshall & Snelgrove and C. & A. Modes (War Damage), Birmingham Technical College (Steel Frame Building) and Grosvenor House, Birmingham (Foundations) at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 7 p.m.

Hon. Secretary : F. G. Fletcher, 60, Brean Avenue, South Yardley, Birmingham, 26.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, December 2nd, 1952

Mr. D. M. Brotton, B.Sc., Ph.D. (Graduate) on " Relaxation Methods," at Middlesborough.

Wednesday, December 3rd, 1952

The above meeting will be repeated at Newcastle.

Tuesday, January 6th, 1953

Joint Meeting with the Institution of Civil Engineers at Middlesborough.

Wednesday, January 14th, 1953

Joint Meeting with the Northern Architectural Association at Newcastle.

Tuesday, February 3rd, 1953

Mr. B. A. E. Hiley, M.I.C.E. (Member of Council) on "Electricity Generating Stations," at Middlesborough.

Wednesday, February 4th, 1953

The above meeting will be repeated at Newcastle.

All meetings will commence at 6.30 p.m., the Middlesborough meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and the Newcastle meetings in the Neville Hall, near the Central Station.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesborough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, December 9th, 1952

Films : "Welded Structures" and "New Tyne Bridge"—kindly lent by Messrs. Dorman Long & Co., Ltd.

Monday, January 19th, 1953

Joint Meeting with the Institution of Civil Engineers, Northern Ireland Association. Mr. Harold E. Sidwell, M.Sc., A.M.I.C.E., on "Reinforced Concrete Building in Brazil," at Queen's University, Belfast, at 5.45 p.m.

Tuesday, February 10th, 1953

Annual Dinner and Social Function. Visit of the President and Secretary of the Institution. At Northern Counties Hotel, Belfast at 6.30 p.m.

All meetings, except where otherwise stated, will be held in the College of Technology, Belfast, at 6.45 p.m., preceded by Tea at the Overseas League premises, Wellington Place, Belfast at 6 p.m.

Hon. Secretary : S. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The following meeting has been arranged :—

Tuesday, December 16th, 1952

Mr. J. H. Huntley on "Structural Design of Cranes," at the Ca'doro Restaurant, Glasgow, at 6 p.m.

Hon. Secretary : D. G. Drummond, B.Sc., M.I.Struct.E. A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, January 23rd, 1953

Mr. Leslie Richardson, A.M.I.C.E. (Associate Member) on "Construction of Two Power Stations in the South West."

Friday, February 6th, 1953

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator) on "Unusual Design for a large Constructional Shop."

All meetings will be held at the Duke of Cornwall Hotel, Millbay, Plymouth, at 7 p.m.

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10-12 Market Street, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Monday, December 1st, 1952

A meeting will be held at Cardiff at which films will be shown.

Wednesday, December 3rd, 1952

A meeting will be held at Swansea at which films will be shown.

Wednesday, January 21st, 1953

Junior Members' Evening at Swansea.

Wednesday, February 11th 1953

Mr. D. Manolopoulos (Member) on "Report on the 4th Congress of the International Association of Bridge and Structural Engineering," at Swansea.

Tuesday, February 17th, 1953

The above meeting will be repeated at Cardiff.

Meetings at Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings in Swansea will be held at the Mackworth Hotel at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, December 5th, 1952

Mr. P. J. Ward on "The Design and Erection of Television Masts."

Friday, January 2nd, 1953

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator) on "Unusual Design for a Large Constructional Shop."

Friday, February 6th, 1953

Mr. F. G. Clarke (Associate Member) on "Some Local Contracts and Welded Steelwork for a Bus Garage."

Wednesday, February 18th, 1953

Annual Dinner.

All meetings, unless otherwise stated, will be held in the University of Bristol Geology Lecture Theatre (entrance University Road) at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 39, Effingham Road, St. Andrew's Park, Bristol, 6.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, December 17th, 1952

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator) on "Unusual Design for a Large Constructional Shop."

Wednesday, January 21st, 1953

Mr. Donovan H. Lee, B.Sc., M.I.C.E., M.I.Mech.E., (Member of Council) on "Design of Prestressed Concrete."

Wednesday, February 18th, 1953

Mr. S. Mackey, M.E., B.Sc., Ph.D., A.M.I.C.E.I., (Associate Member) on "Secondary Stresses in Steel Bridge Girders."

All meetings will be held at The University, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone: 34-1111 Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.